FEBRUARY 1961

Journal

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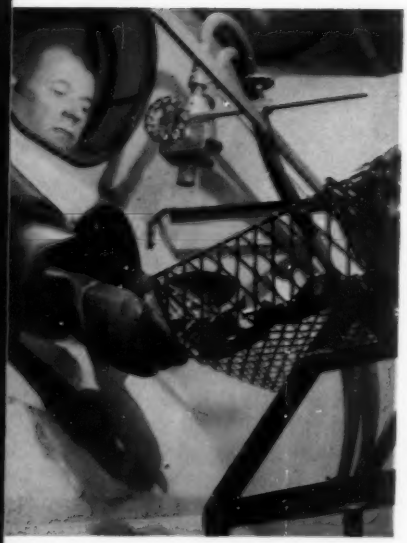
AMERICAN WATER WORKS ASSOCIATION



Glass-blasting chambermaid doing an inside job on messy meters at Long Beach

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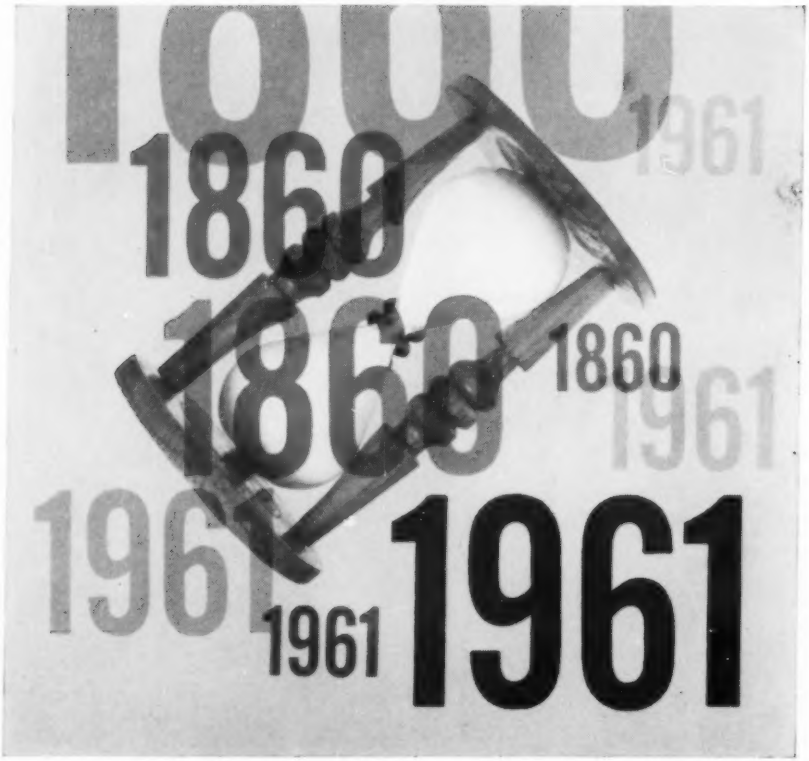
Detroit, June 4-9



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Detroit, Mich. June 4–9, 1961

Official reservation forms mailed to all members are returnable to AWWA postmarked on or after (but not before) noon, Feb. 21

Coming Meetings

AWWA SECTIONS

Spring 1961

Mar. 16—New England Section, at Tufts University, Medford, Mass. Secretary, Ralph M. Soule, San. Engr., State Dept. of Public Health, Boston, Mass.

Mar. 22–24—Illinois Section, at LaSalle Hotel, Chicago. Secretary, Dewey W. Johnson, Research Engr., Cast Iron Pipe Research Assn., 3440 Prudential Plaza, Chicago 1.

Apr. 5–6—West Virginia Section, at West Virginian Hotel, Bluefield. Secretary, Hugh W. Hetzer, Design & Construction Dept., Union Carbide Chemicals Company, Box 8361, South Charleston.

Apr. 6–8—Montana Section, at Placer Hotel, Helena. Secretary, A. W. Clarkson, Asst. Director, Div. of Environmental Sanitation, State Board of Health, Helena.

Apr. 11–13—New York Section, at Statler-Hilton Hotel, Buffalo. Secretary, Kimball Blanchard, New York Branch Sales Office, Neptune Meter Co., 22–22 Jackson Ave., Long Island City 1.

Apr. 12–14—Kansas Section, at Baker Hotel, Hutchinson. Secretary, Harry W. Badley, Representative, Neptune Meter Co., 119 W. Cloud, Salina.

Apr. 14—California Section, at Biltmore Hotel, Santa Barbara. Secretary, Frank F. Watters, Hydr. Engr., State Public Utilities Com., State Bldg., Civic Center, San Francisco.

Apr. 19–21—Nebraska Section, at Cornhusker Hotel, Lincoln. Secretary, Joseph J. Rossbach, Metropolitan Utilities, 18th & Harney Sts., Omaha.

Apr. 20–22—Arizona Section, at San Marcus Hotel, Chandler. Secretary, A. D. Cox, Jr., Secy. & Comptroller, Arizona Water Co., Box 5347, Phoenix.

Apr. 23–26—Southeastern Section, at Poinsett Hotel, Greenville, S.C. Secretary, N. M. deJarnette, 96 Poplar St., N.W., Atlanta, Ga.

(Continued on page 8)

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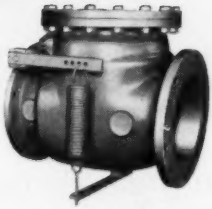

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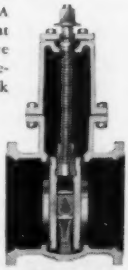

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Coming Meetings

(Continued from page 6)

Apr. 26–29—Pacific Northwest Section, at Empress Hotel, Victoria, B.C. Secretary, Fred D. Jones, W. 2108 Maxwell Ave., Spokane, Wash.

Jun. 1–3—Canadian Section, at Prince Edward Hotel, Windsor, Ont. Secretary, A. E. Berry, 72 Grenville St., Toronto, Ont.

Jun. 20–22—Pennsylvania Section, at Galen Hall Hotel, Wernersville. Secretary, L. S. Morgan, 413 First National Bldg., Greensburg.

Fall 1961

Sep. 11–13—Kentucky-Tennessee Sec., Louisville, Ky.

Sep. 13–15—New York Sec., Saranac Lake.

Sep. 13–15—North Central Sec., Minneapolis, Minn.

Sep. 20–22—South Dakota Sec., Rapid City.

Sep. 27–29—Wisconsin Sec., Milwaukee.

Oct. 1–3—Missouri Sec., Springfield.

Oct. 4–6—Virginia Sec., Roanoke.

Oct. 8–11—Alabama-Mississippi Sec., Biloxi, Miss.

Oct. 15–15—Southwest Sec., San Antonio, Tex.

Oct. 18–20—Iowa Sec., Cedar Rapids.

Oct. 25–27—California Sec., Sacramento.

Oct. 25–27—Ohio Sec., Toledo.

Oct. 25–28—New Jersey Sec., Atlantic City.

Oct. 29–Nov. 1—Florida Sec., Orlando.

Nov. 1–3—Chesapeake Sec., Baltimore, Md.

OTHER ORGANIZATIONS

1961

Feb. 26–Mar. 2—American Institute of Mining, Metallurgical & Petroleum Engineers, Chase-Park Plaza Hotel, St. Louis, Mo.

Mar. 13–17—National Assn. of Corrosion Engineers, Statler Hotel, Buffalo, N.Y.

Mar. 30–31—Industrial Water & Waste Conference, Rice Univ., Houston, Tex., sponsored by the Texas Water & Sewage Works Assn. Write: Professor A. W. Busch, Rice Univ., Houston.

Apr. 6–7—10th Southern Municipal and Industrial Waste Conference, Duke University, Durham, N.C. Write: Edward H. Bryan, Dept. of Civil Engineering, Duke Univ., Durham.

May 2–4—16th Annual Industrial Waste Conference, Memorial Center, Purdue Univ., Lafayette, Ind. Write: Don E. Bloodgood, Prof. of San. Eng., Purdue Univ., Lafayette.

Jun. 25–30—ASTM Annual Meeting, Atlantic City, N.J.

SHORT COURSES

1961

Feb. 20–24—Course on "Radionuclides in Water," R. A. Taft Sanitary Engineering Center, Cincinnati, Ohio. Write: Chief, Training Program, 4676 Columbia Pkwy., Cincinnati 26, Ohio (or to USPHS regional office).

Feb. 27–Mar. 10—Course on "Water Quality Management—Sanitary Engineering Aspects," R. A. Taft Sanitary Engineering Center, Cincinnati, Ohio. Write: Chief, Training Program, 4676 Columbia Pkwy., Cincinnati 26, Ohio (or to USPHS regional office).

Mar. 15–17—24th Annual Short Course, Engineering Experiment Station, Louisiana State Univ., Baton Rouge 3, La.

Apr. 5–7—Symposium on ground water contamination, Sheraton-Gibson Hotel, Cincinnati, Ohio. Write: Director, Robert A. Taft Sanitary Engineering Center, 4676 Columbia Pkwy., Cincinnati 26, Ohio.

Apr. 23–28—Training course on "Environmental Health Aspects of Health Mobilization," OCDM Staff College, Battle Creek, Mich., sponsored by the USPHS Div. of Health Mobilization. Write: Mrs. Jean M. Nowak, Information Officer, Div. of Health Mobilization, USPHS, Washington 25, D.C.

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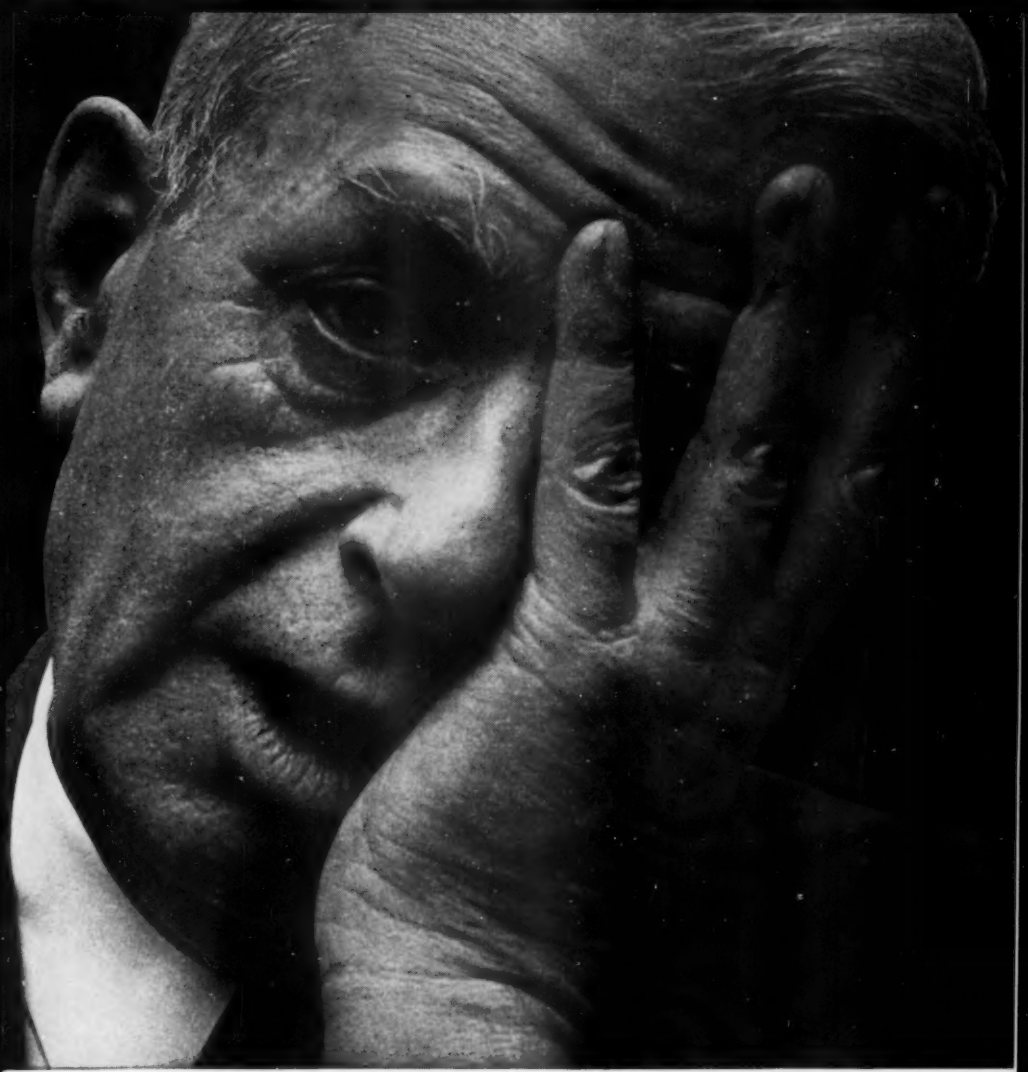

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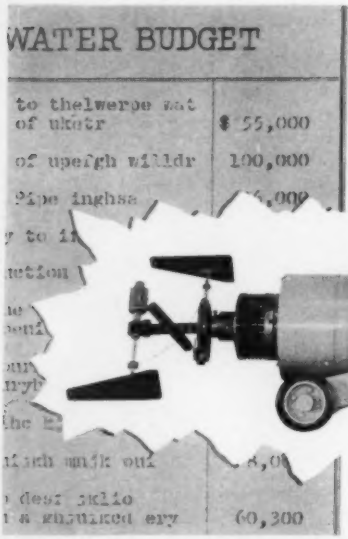

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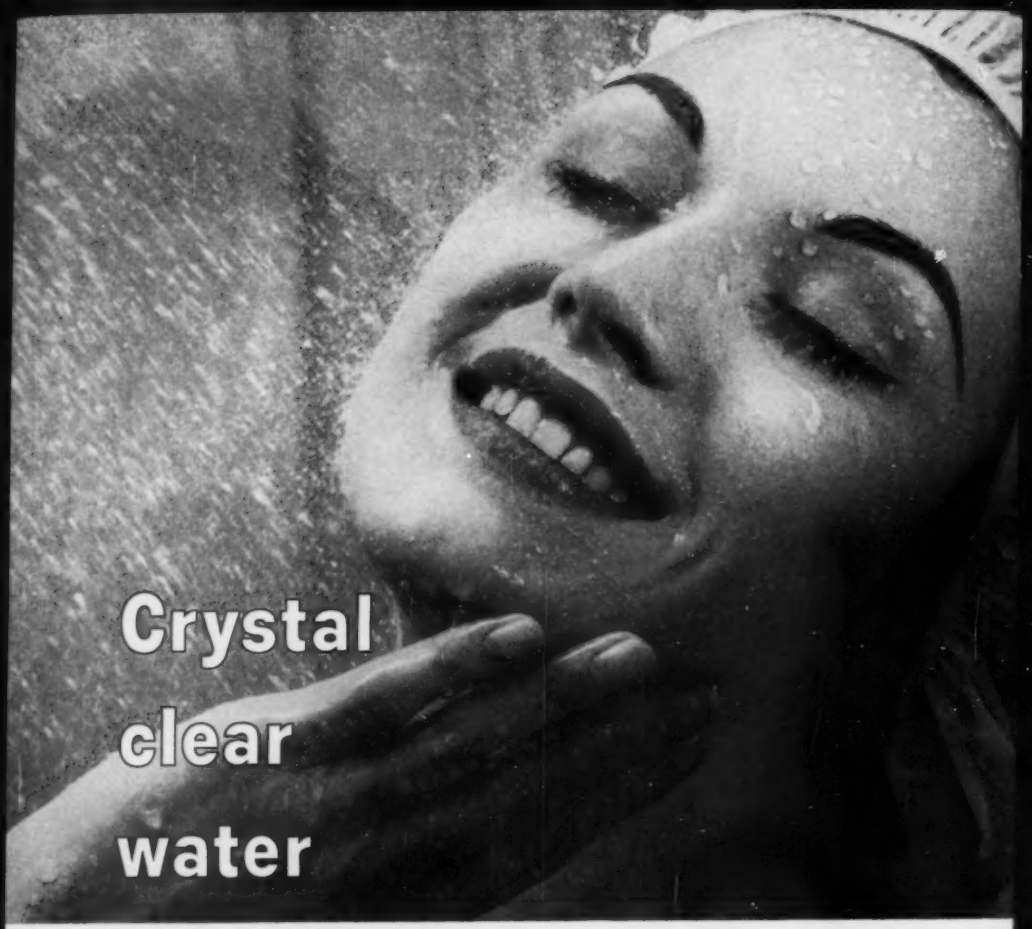

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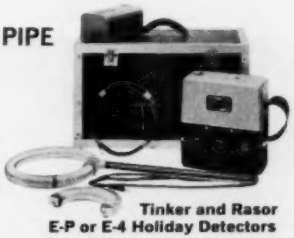

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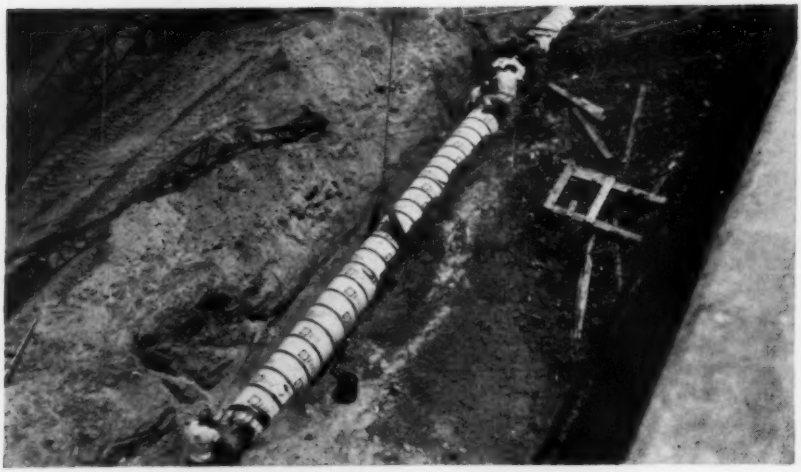

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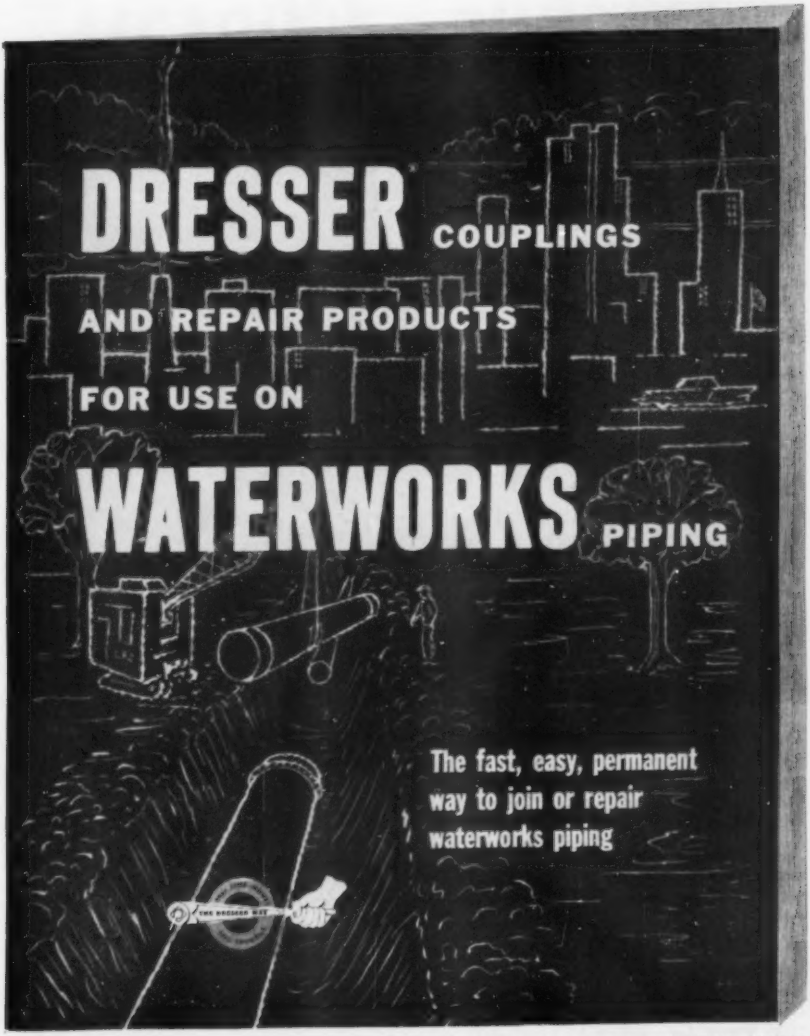

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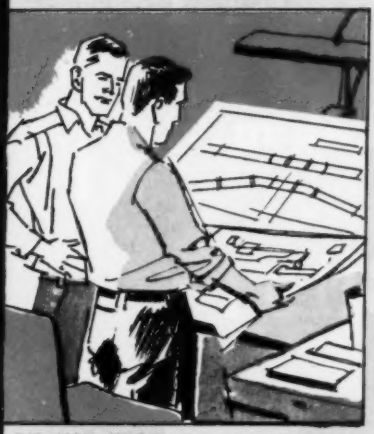

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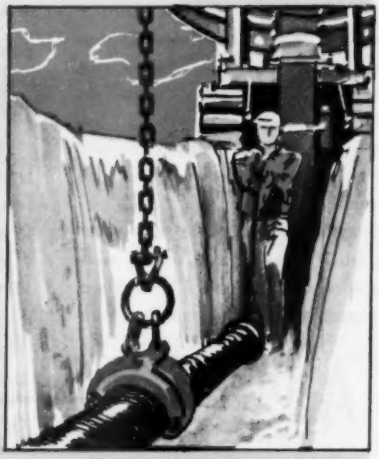

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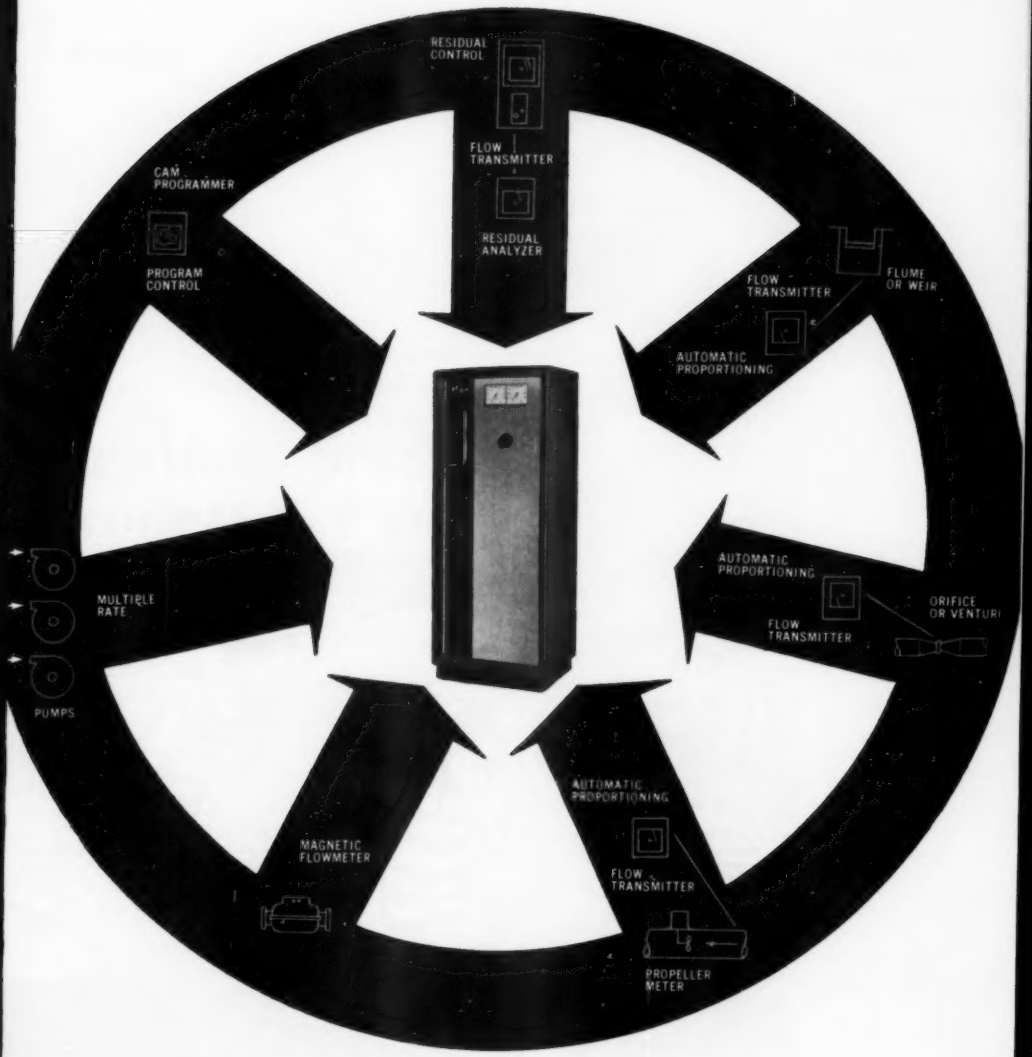

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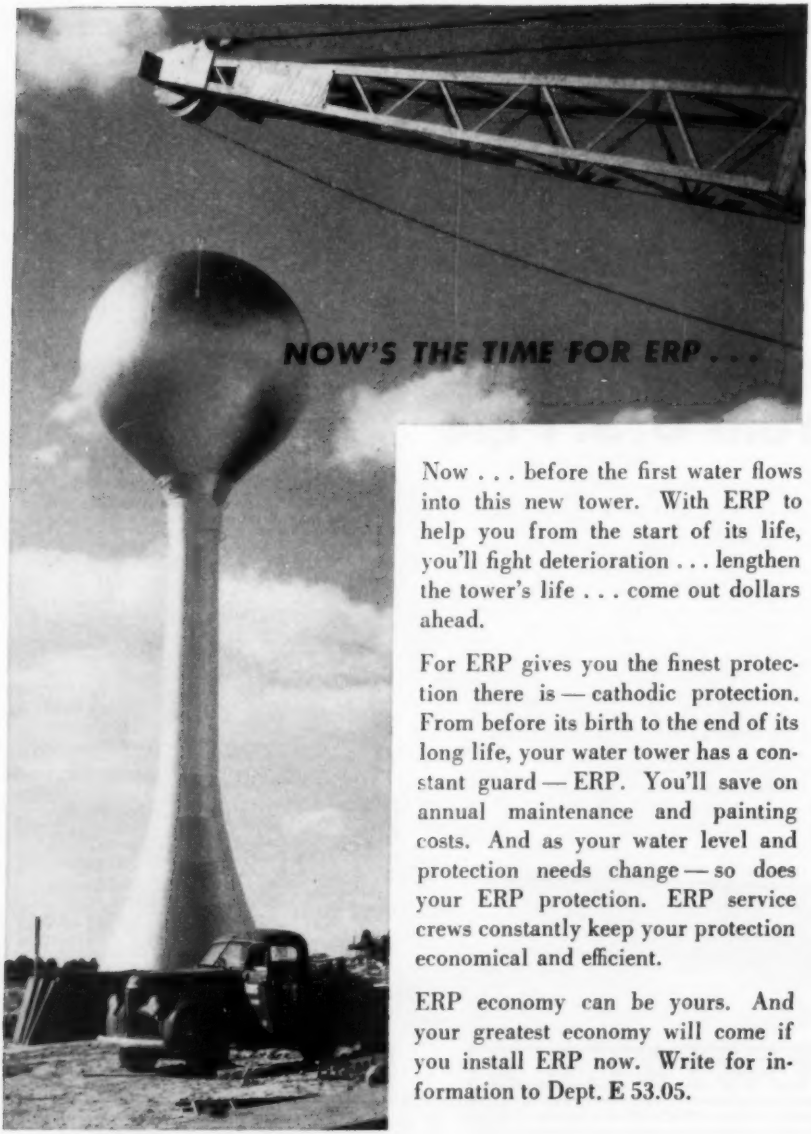

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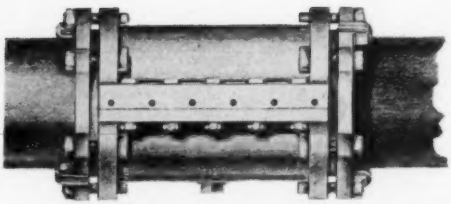
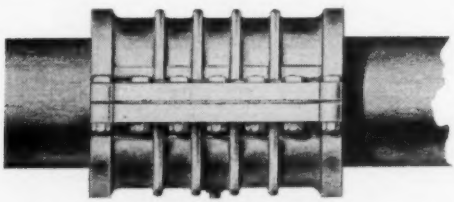


Mechanical Joint Repair Sleeve



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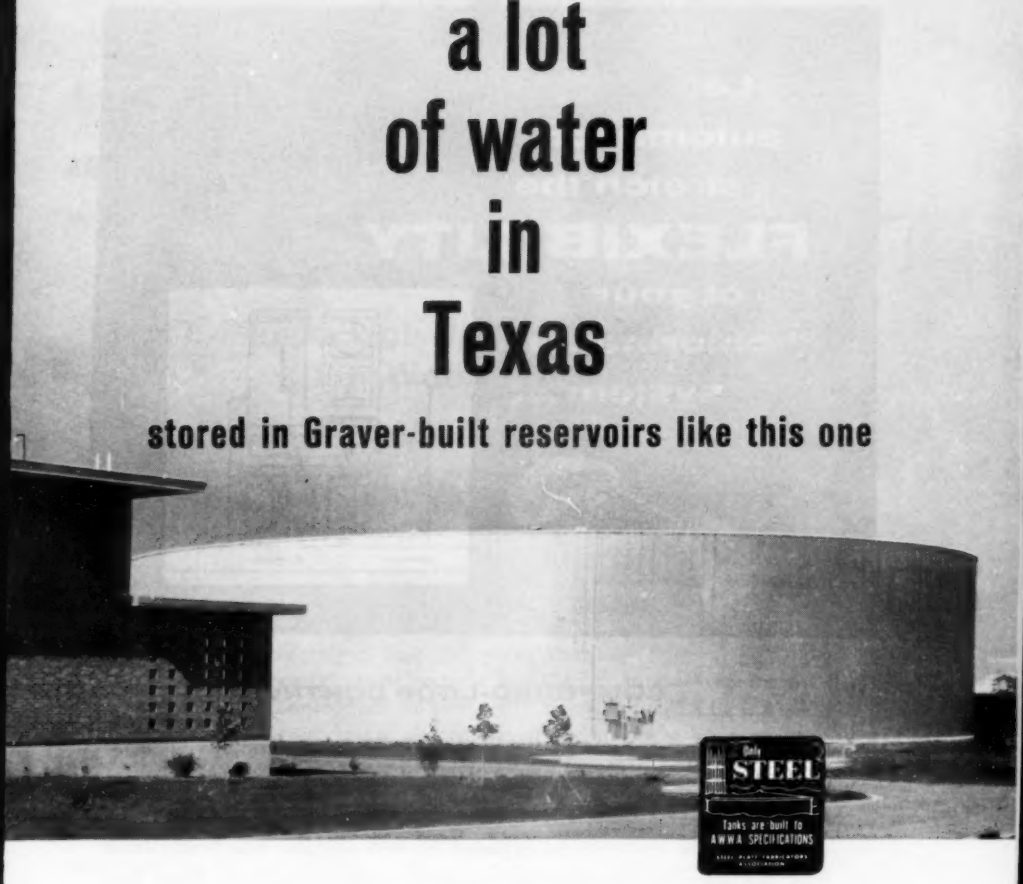

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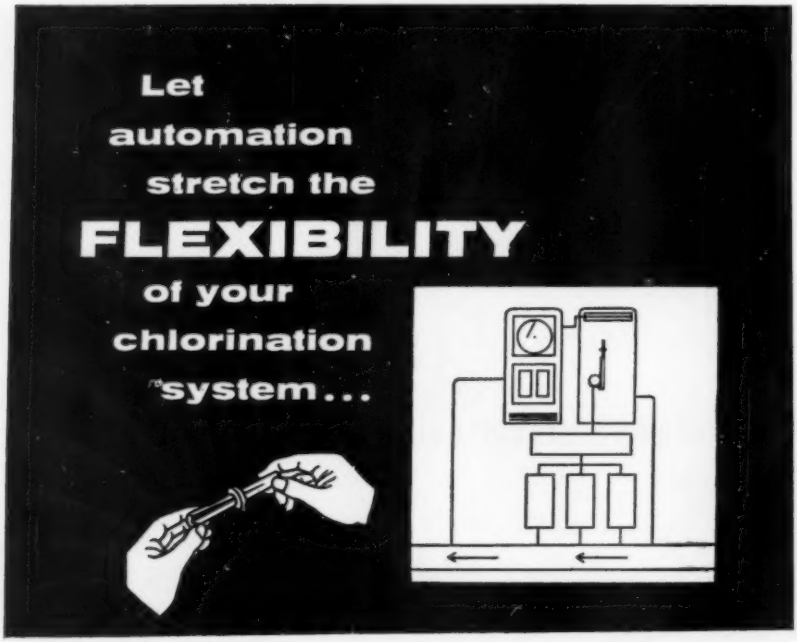

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Journal

AMERICAN WATER WORKS ASSOCIATION

VOL. 53 • FEBRUARY 1961 • NO. 2

Impact of Desalinization on the Water Economy

Abel Wolman

A paper presented on Oct. 14, 1960, at the Eastern Water Co. Conference, Atlantic City, N.J., by Abel Wolman, Prof. of San. Eng., Johns Hopkins Univ., Baltimore, Md.

THE interest in desalinization developed during the last 5 years stems from the general assumption that the United States and many other parts of the world will be confronted in less than a decade with severe water shortages. It is important, therefore, as a setting for any discussion of the impact of desalinization on the water economy, to determine whether the predicted restrictions on the availability and use of water are in fact as distressing as many people seem to believe.

It has been generally estimated that, by 1980, the total withdrawals from streamflow in the United States for purposes of irrigation, manufacturing, mining, steam electric power, and municipal uses will total approximately 560 bgd. The major withdrawals, of course, will be for irrigation, manufacturing, and steam electric power. So that this figure should not cause too much alarm, it should be remembered that the major part of these withdrawals are returned to the streams for subsequent reuse. More accurate and less alarming figures indicate that the actual consumptive use in this country will rise to only 120 bgd in 1980. This figure, incidentally, will represent somewhat less than 11 per cent of the average streamflows available.

One should not forget, however, that water availability is a function of place, time, and climate. Average figures, therefore, tend to conceal the fact that some areas in the United States will encounter water difficulties earlier and to a greater degree than others. Desalinization, hence, does have a significance, the magnitude of which will be discussed later. It is extremely doubtful that scarcity of water on a broad basis, at least in the United States, will impose a serious restriction on the nation's ultimate growth.

Competition for water in the open market will undoubtedly parallel that of the past. Existing supplies may be

diverted from current uses to higher-value uses. Some of this evolution has already taken place, of course, in the shifting of water from irrigation to industrial uses in many parts of the United States.

Production Costs

Much loose talk has been indulged in with respect to desalinization processes. The methods of removal of soluble minerals of various concentrations in water have not been sharply delineated in much of the literature. This confusion regarding concentrations of salts and methods of desalinization or demineralization has resulted in difficulty in defining actual costs.

TABLE 1

Characteristics of Mineralized Waters

Description of Waters	Concentration of Dissolved Solids *ppm*
Brackish	1,000–3,000
Moderately saline	3,000–10,000
Sea water	33,000–36,000
Brine*	36,000+

* As found in the Dead Sea and Salt Lake, for example.

For purposes of this discussion, the classification of waters shown in Table 1 is adopted.

Several years ago official agencies in the federal government suggested that production costs of desalinization processes should aim to reach certain price goals—namely, about 38 cents per 1,000 gal for municipal or domestic waters and 12 cents per 1,000 gal for irrigation water uses. Such goals were to be compared with production costs of desalinization, without the additional cost for distribution to the points of actual use.

In recent years, less and less emphasis is being placed officially on these goals, because they have become somewhat embarrassing, particularly in relation to agricultural uses. This embarrassment arises from the fact that, although the production cost of desalting or demineralizing water has declined appreciably in the last 3 or 4 years, it is improbable that the figure quoted above for irrigation use will be reached in the foreseeable future.

When production costs of desalinization noted later herein are considered for comparison with fresh-water costs, it should be emphasized that the use of desalted water in general will: (1) necessitate higher pumping lifts to the consumer from sea level elevations, (2) not include the cost of distribution and multiple repumping into service zones, and (3) provide for no filtration or other treatment costs, no cost of waste disposal, no holdover storage to meet the variable demands of consumers, and no research or development costs.

Sea water conversion. Information now available on sea water conversion indicates clearly that the bulk of these operations is generally accomplished by one or another form of distillation. About sixteen distillation plants for sea water conversion are now in operation throughout the world, with capacities of 0.1–3.5 mgd. The plants have an aggregate capacity of somewhat less than 15 mgd.

At the Aruba plant on the Caribbean Sea, the cost of production per 1,000 gal is apparently between $1.25 and $1.75. In the Morro Bay (Calif.) power plant of the Pacific Gas & Electric Co., the cost of desalinization per 1,000 gal is $2.50, when the plant is running at full capacity. In Kuwait, adjoining Saudi Arabia, the Kuwait Oil Co. is producing fresh water from Arabian Gulf sea water by evaporation. Unfortunately, no cost data are avail-

able for this plant, because of the major difficulties in bringing it into complete and successful operation. The costs are undoubtedly in excess of $1 per 1,000 gal. At the Oxnard plant of the Southern California Edison Co., a pilot plant for desalting sea water by flash evaporation is in operation. No definitive costs for this plant's operation are yet available.

Even with these limited observations, one might conclude that the production costs in the late 1930's of $4–$5 per 1,000 gal have been materially reduced in actual operation to approximately $2. It is the hope of the official agencies in the federal government dealing with these operations that newer installations converting sea water to fresh water will result, in the next 5 or 10 years, in a production cost of about $1 per 1,000 gal or $326 per acre-foot.

Demineralization of brackish waters. A number of methods have recently come into use for the removal of soluble mineral salts from waters containing very small fractions of the salt concentrations found in sea water—namely, concentrations between 1,500 and 2,500 ppm of soluble salts. The work done with South African mine wastes is frequently noted as desalting of sea water, but, of course, it is no such thing. The raw water falls within the definition of brackish water. The successful application of membrane processes to demineralizing mine wastes in South Africa is resulting apparently in production costs of somewhat less than 50 cents per 1,000 gal.

The Coalinga, Calif., experience with demineralization has also been confused with the desalting of sea water. The Coalinga plant is demineralizing water of ground water origin that has a concentration of dissolved salts somewhat less than 2,500 ppm. The present production cost at the plant is $1.43 per 1,000 gal. This is compared with the former cost of carting water into the community at $7.05 per 1,000 gal. The saving is obviously great. Further experience with the process may result in reducing the production cost to less than 80 cents per 1,000 gal. In Bahrein, a similar process is in operation, with an estimated cost of production, at full plant capacity, of $1.27 per 1,000 gal.

It is difficult to predict the ultimate costs resulting from the processes of demineralization of brackish water. It is not overly optimistic to assume that, with better and better equipment and longer and longer membrane life, the production costs should approach 75 cents per 1,000 gal ($245 per acre-foot) or less. It should be mentioned again that the foregoing costs do not include costs of waste disposal and costs of distribution to the consumer, both of which represent a considerable sum.

Other processes for demineralization or for true desalting of sea water are for the most part still in the pilot plant stage. Any effort to predict their probable production cost would certainly be premature, necessitating extrapolation from the production of 15,000 gpd for a pilot plant process to 5–10 mgd for a major process. Some believe that conversion costs with freezing processes may, for brackish water, approach $1 per 1,000 gal, although these processes are still only in the laboratory or pilot plant stage.

Economic Comparisons

When one compares the present and prospective production costs for supplying fresh water and demineralized water, it should be recalled that the

consumer will normally choose those waters that are cheapest, quality being equal. In areas where fresh water is not easily made available, recourse will be had, if the necessity arises, either to brackish water or sea water, if these in turn are at hand. The water becomes available when it is worth producing for particular purposes. Choice of source and payment for it then become in fact true economic decisions.

For purposes of comparison, therefore, one must review the usual costs of water production by municipalities and industries. Many of these costs are available in considerable detail, and their exploration is not undertaken here. It may be said, however, with considerable accuracy, that industrial and municipal production costs rarely exceed 25 cents per 1,000 gal. In the Los Angeles area, fresh-water production costs average 3–20 cents per 1,000 gal, or $10–$70 per acre-foot, even with the very long aqueduct delivery systems there installed. In the Baltimore area, the production cost of municipal water is less than 10 cents, exclusive of filtration and distribution costs. In some instances elsewhere, the actual delivery cost of fresh-water supplies, including all aqueduct costs but excluding treatment, is less than 20 cents per 1,000 gal. Even the relatively expensive Colorado River water produced by the Metropolitan Water District of Southern California, necessitating a total pump lift of 1,600 ft, costs less than $30 per acre-foot delivered.

It is striking that the cost of water at the Delta with the California multiple-purpose water program is only 1.075 cents per 1,000 gal ($3.50 per acre-foot). S. B. Morris,[1] in this connection, pointed out that the cost of irrigation water delivered to the farmer's headgate is generally $1–$10 per acre-foot (0.307–3.07 cents per 1,000 gal). For limited areas of very high-value crops, this cost may be as much as $40 per acre-foot (12.28 cents per 1,000 gal).

Future Prospects

The data presented here make it clear that it would be a grave mistake to hope to find an early economic solution to water problems in the desalinization of sea water, so that desalted water becomes a major competitor of large fresh-water supplies. With respect to irrigation in any feasible fresh-water project, it is equally clear that such a project cannot, according to available evidence, be economically challenged by any demineralizing process now known. Even in the Pacific Coast states, where water shortages have been overemphasized in the public press, it is clear that fresh water can be made available for requirements after 1990 at a cost that is very low.

TABLE 2

Results of Water Conservation in Industries

Industry	Quantity of Water Used Before Conservation *gal./ton of product*	Quantity of Water Used After Conservation *gal./ton of product*
Steel	55,000	1,500–3,000
Paper	7,000–10,000	3,000–4,000
Kraft pulp	45,000	9,000
Newsprint	26,000	6,000
Cannery	2,500	1,200

Morris [1] stated that "it is not enough to point out the increasing demands on fresh water supplies and suggest that future shortages, when they occur, may be met by extracting fresh water from the sea and inland saline waters." He

indicated that, instead, a more specific approach, weighing local needs and ability to pay, with alternative sources of water available, is necessary.

The author has estimated that by 1965 the worldwide use of land-based sea water distillation plants may account for as much as 50 mgd of fresh water. Those plants used for the conversion of brackish waters may account for an additional 20 mgd. When these amounts are compared with the billions of gallons necessary throughout the world, it is conservative to assume that the total effect of desalinization and demineralization on the national and international water economy will be minor.

But before he is accused of standing in the way of progress, the author hastens to emphasize the great importance of such conversion processes in many scattered areas of the United States and elsewhere. In those regions, both brackish water and, to a lesser extent, sea water may be converted, simply because no cheaper or other sources of water are available. These facts point to the undoubted desirability of pursuing, with increased energy and money, research in the basic physical chemistry of highly mineralized water. It is gratifying that the federal government has recently recognized that only minimum amounts of money have been spent hitherto in such research and has provided additional funds for the immediate future. Of equal importance is the necessity for moving forward with rapidity in engineering design and development in order to continue to reduce the very wide gap between the theoretic minimum power requirements for desalinization processes and the actual plant accomplishments.

Alternatives to Desalinization

In a discussion of the competitive costs of desalinization and fresh-water production, one cannot escape the conclusion that, where fresh water becomes scarce, cheaper alternatives to desalinization or demineralization are often available. These alternatives usually come into play when water is difficult to get, costs for development rise to uneconomic levels, and geographic proximity favors reuse of waste waters. Examples of such circumstances are in Aruba and Southern California and at the Bethlehem Steel Co. in Maryland.

It should be remembered, of course, that the reuse of waste waters is practiced nearly throughout the world. That it will be expanded is undoubted, as improved and cheaper modes of waste treatment and water treatment are developed. Here is a most fruitful field for research and development.

Elaboration of the possibilities of reuse as a major future source of required water is reported in detail in a document [2] submitted to the Senate Select Committee on Water Resources. A few examples are culled from this document to illustrate the potentialities of reuse already mentioned:

1. The Bethlehem Steel Co. at Sparrows Point, Md., employs direct reuse, for industrial purposes, of completely treated sewage plant effluent from Baltimore. From 1942 to 1960, the amount of reused effluent has risen from 25 mgd to 115 mgd. Cost of delivery, including fixed charges on two pipelines 26,000 ft in length, pumping stations, and treatment plants, is approximately 3.5 cents per 1,000 gal.

2. With regard to savings of water for industrial use, the data shown in

Table 2 give some indication of the effect of conservation when its practice has become profitable for industry.

3. Use of irrigation water per acre of land in the United States is often excessive. The Department of Agriculture estimates that, by the year 2000, savings of water due to more efficient irrigation practices may be as much as 50,000,000 acre-ft/year.

4. Control of useless phreatophytes (water-loving plants) may save about 6,000,000 acre-ft of water per year.

5. Underground storage of surface waters that now go to waste is recommended.

6. Sea water contains about 35 times as much dissolved solids as does sewage. The removal of dissolved solids from sewage is undoubtedly much cheaper than that from sea water.

7. Many examples of the use of sewage plant effluents for irrigation purposes are available. Such use must be controlled, with careful attention given to the public health implications. A most recent example, although it is on a relatively modest scale and after about 6 years of experimentation, is in the Talbert Valley in Southern California. The farms use the liquid from the primary sewage treatment plant of the Orange County Sanitary District. About 2,800 acres are now being irrigated from this source, at a cost of about $6 per acre-foot.[3]

None of these examples produces *additional* water, but they have great significance in stretching the nation's water resources over a greater population, if and as required.

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Water, Salt, and Survival

James Bonner

A contribution to the Journal by James Bonner, Prof., Div. of Biology, California Inst. of Technology, Pasadena, Calif.

NOWADAYS there is much discussion about taking the salt out of sea water so that man can use the water. Actually, this is an odd thing to have to worry about; after all, it is believed that all life originated in the oceans, that land-dwellers only gradually, and at the expense of vast exertion, made their exodus from the sea. Now, after a long process of adaptation, they are unable to survive in their ancestral haunts.

Man and Fresh Water

Man has the upper hand in the world today, but if he is to maintain himself and his industrial culture, he must have an ample supply of fresh water. He depends on a diet composed almost entirely of the produce of cultivated plants, and cultivated plants are notably intolerant of salt. As a matter of fact, it has been suggested that one way to circumvent the problem of finding fresh water to support land-bound agriculture might be to develop, instead, an agriculture in which marine algae would replace the more familiar crop plants. This is already being done in Japan, where seaweed is an important part of the national diet. For a number of reasons, however—among them economic considerations—the majority of mankind will probably turn to saline-water conversion to produce the needed fresh water.

Because the removal of salt from sea water is becoming such an important topic in the water industry, it might be worth while to investigate some of the biologic aspects of water, salt, and survival.

Importance of Salt

Land animals and land plants lose water by several means, and because there are salts in their diets (and there must be, as they need a certain minimum amount of various ions), they would continuously get saltier and saltier if they were unable to rid their bodies of excess salts. Land animals excrete urine, which is more concentrated in salt than their diet or their blood. If their diet is too salty, however, they cannot excrete a solution more concentrated in salt than their diet, and they expire.

It is interesting to consider the difference between the problem that confronts an animal that lives immersed in fresh water and one that lives in salt water. A fresh-water animal must keep its internal salt concentration higher than the external concentration. It must accumulate salt to achieve the concentration—not the amount—of ions that is necessary to its survival. This is difficult, as nature keeps trying to establish osmotic equilibrium by leaking water into the organism and diluting it. The animal has to do something about it and does, by excreting urine, or its equivalent, which is a more dilute salt solution than its body contents—that is, it se-

lectively excretes water. In a marine animal, the problem is that, although the internal salt concentration must be kept lower than the external concentration, salts continually leak in. Thus the organism must excrete a fluid, urine or its equivalent, which is more concentrated in salt than its body contents—that is, it must selectively excrete salt. These, then, are the ways in which creatures adjust themselves to the salt concentration of their environments.

Ion Transport

As everyone knows, all living things are composed of cells. The cells are surrounded by special membranes that have the property of semipermeability. The cell membranes, which enclose the living protoplasm, are in general very permeable to water molecules. Water molecules dash in and out of living matter with great facility. The cell membranes are also permeable to certain non-ionic solutes. Such non-ionic solutes permeate living material and water passes in and out until the concentration of solute inside the cell is in equilibrium with that outside the cell.

Protoplasmic membranes are in general much less permeable to non-ionic solutes than they are to water, however, and the equilibration times for such organic molecules as sugar or alcohol are often very long. So far as is known today, protoplasmic membranes have no true permeability to ions.

Ionic materials do not move across protoplasmic membranes and into cells by any ordinary type of diffusion mechanism. Instead, the movement of ions into cells is accomplished by a process that biologists call "active transport," and it is dependent upon a continuing supply of energy provided by the cell's own respiratory metabolism. In fact, the ion may be concentrated by this transport mechanism until the concentration of ion inside the cell far exceeds its concentration outside the cell. Thus, chemically speaking, the mechanism of ion uptake is highly unorthodox.

So far as is known today, an ion, once it has been taken into the cell, does not leave. The permeability of the cell membrane for outward diffusion of ions out of the cell is just as small as for inward diffusion. Furthermore, there is not just one transport mechanism, but several. Different species of ions have their own specific transport mechanisms for entering into living material. Potassium ions, for example, have a transport system different from that of sodium ions. Sodium and potassium, therefore, do not compete with one another but enter independently. Thus a cell can accumulate sodium and potassium at different rates even though the two ions are present in the external solution in identical concentration.

It is known that for plants, in which the matter has been most extensively studied, there is one transport system for the ions sodium and lithium; a second and independent system for the ions potassium, rubidium, and cesium; a third for calcium, strontium, and barium; a fourth for magnesium; a fifth for sulfate and selenate; a sixth for phosphate and arsenate; a seventh for the halide ions; and so on.

Each transport system seems to involve a carrier, a compound, which appears at the external surface of the cell, chelates with or otherwise complexes the ion to be carried, moves it to the inside of the protoplasmic membrane, and is there changed by the respiratory metabolism in some appropriate way so that the ion is freed

and the carrier regenerated to return to another cycle of activity. The important task of discovering exactly how ion transport works on the molecular level has not been completed, however. Indeed, it has scarcely been begun.

Finally, it is of interest that the transport systems for ions have their own special kind of concentration dependence. Rate of salt uptake by a cell increases with external salt concentration up to some maximum rate which is achieved generally at concentrations of the order of 10 millequivalents per liter or less of the ion. At higher salt concentrations, salt uptake remains constant. Thus, the salt transport system is saturated at rather low salt concentrations.

Once taken up by the cell, the salts are in part dumped into appropriate depots within a cell and in part transported by the circulatory system and dispersed throughout the organism. In a plant, for example, salts are carried up to the leaves. As the plant continuously loses water by evaporation, salts are continuously concentrated in the leaves. This is doubtless one of the principal factors which result in the ultimate death of the leaves of plants!

Biologic Ion Requirements

The reason that organisms have elaborate arrangements for the accumulation of salts is that all living things need certain inorganic ions. Plants do not need sodium, however, or at least they need it only in the most minute quantities. They also do not need very much chloride. They are equipped with an ion accumulation system so that they can concentrate the needed ions to the required level even though the water or soil solutions from which they feed are very low in these ions. At the same time, no living cell can stand too high a salt concentration, too high an ionic strength. The vital processes which require ionic interaction cannot take place if the total ionic strength is too great. Neither can a living cell tolerate an unduly high concentration of any one kind of ion—sodium, for example. Each ion has its own special duties, its own special place to sit, and sodium cannot take the place of potassium or magnesium. Thus the ionic strength and the ionic composition of a living cell must remain within certain rather narrow limits in order for the cell to live.

Salt Tolerance

Many land creatures, including the domestic plants upon which human beings depend, possess a very low requirement for sodium and, in fact, find sodium obnoxious. They have arrangements to let only a small quota of sodium into the cell. Thus when sodium and chloride ions are present in the external solution, they act as osmotically active solutes. When placed in a solution containing a high concentration of sodium chloride, a crop plant is unable to adjust its internal osmotic concentration of salt to a level sufficiently greater than the external concentration and, after a certain point, cannot take up any water at all. For this reason, if for no other, the ordinary everyday land plants cannot stand water which is too salty. Sodium chloride is to them an osmotically active external solute.

Actually, the tolerance to salt of land plants—and, indeed, of land animals—varies over a wide range. The brine shrimp lives happily in a 27 per cent salt solution in Great Salt Lake, whereas the alga which is responsible for the red patches on the surface of fresh snow cheerfully resides in near-distilled water.

Among the land plants, two extreme types can be distinguished. One type is intolerant of salt in any concentration more than a few millequivalents per liter. The second type—the halophytes—is tolerant of salts, and specifically of sodium chloride, in substantial concentrations (from a few tenths to one equivalent per liter).

Halophytes are land plants that have compromised. They accumulate sodium and chloride just as happily and readily as they accumulate potassium or sulfate. For the halophytes sodium chloride is not an osmotically active external solute; it is just another solute that enters with the other ions and is concentrated in the cell interior. In the halophytes, total cellular ionic strength can rise to very high levels indeed. But although these plants can tolerate high external osmotic concentrations and high sodium chloride concentrations, they do so at a price. They are pretty poor plants—slow growing, of low photosynthetic rate, with shabby enzymes. Like *Artemia salina*, the brine shrimp of Great Salt Lake, the halophytes are able to live in salty places, but they cannot compete with normal plants in nonsalty places.

Although man does not know all about salt and biology, enough is known to predict with some assurance that it will be difficult to breed cultivated plants or domestic animals which are much more tolerant of salt than those which are known today. The possibility of breeding the salt-tolerant features of the halophyte into domestic crops appears to be vanishingly small.

Water and Agriculture

The author and his colleagues have worried a great deal about the earth's growing population and how this population is to be supplied with food in the generations to come. It is already known that the availability of fresh water is a limiting factor in the production of food over much of the temperate zone. At Cornell University, for example, in experiments to determine the ultimate maximum yield of crop plants, it was disclosed that even in rainy New York state available water becomes the limiting factor as soon as man does all the other things he knows how to do, such as supply enough of the varied mineral elements that their availability ceases to be a limiting factor.

Although a great deal has been said about how crop plants are enormously wasteful of water—that the plant is just the wick through which the water of the soil is moved up and boiled out into the atmosphere—most biologists agree that loss of water from the leaves is an inevitable consequence of the structure of the leaf, which is dedicated to the ready exchange of carbon dioxide and oxygen, the gases which are the metabolites of photosynthesis.

Thus it appears that man will just have to supply more and more fresh water for agriculture and that this, in the end, will be the most economically feasible route for the increase of total world food supplies—more economical, for example, than the harvesting of marine algae or the growing of algae in closed containers. It is the author's prediction, therefore, that the reclamation of sea water will have, in years to come, an increasingly important role in the feeding of the world's population, and that a few generations from now it may well be commonplace to irrigate the large arid regions of the earth with reclaimed ocean water.

Removal of ABS From Heavily Polluted Waters

Charles E. Renn and Mary F. Barada

A contribution to the Journal by Charles E. Renn, Prof., and Mary F. Barada, Research Assoc., both of the Dept. of San. Eng. & Water Resources, Johns Hopkins Univ., Baltimore, Md.

THERE are a number of problems in converting any deteriorated raw water to a potable supply, but this is particularly true in instances of drought, low flows, emergency reuse, and gross pollution of well supplies found to contain foaming concentrations of synthetic detergents. Substantial concentrations of synthetic detergents are required to produce even a slight foam in natural raw waters. The occurrence of such concentrations usually means that high concentrations of other waste materials and the conditions associated with them also prevail.

It is essential that any emergency regime for removing foaming concentrations of synthetic detergents from raw waters be compatible with other measures used in routine water treatment, especially with those available for handling the emergencies of low flow and gross pollution. Fortunately, this is possible.

AASGP Research Project

During the past 2 years the Association of American Soap and Glycerine Producers (AASGP) has sponsored a reseach investigation of the effectiveness of conventional water treatment processes for the removal of alkyl benzene sulfonate (ABS) from raw water supplies. ABS is the most widely used surface-active component of the ordinary detergents in general domestic use. The work has been carried out in the laboratories of the Department of Sanitary Engineering and Water Resources, Johns Hopkins University.

Activated-Carbon Treatment

Of the various processes available for routine or emergency water treatment, the one that is most immediately adaptable and effective is activated-carbon treatment. An extended series of explorations has shown that the hydrophobic chain in ABS can be more effectively bound by adsorption than the hydrophilic radical. This is why activated carbon is preferable to aluminum, iron, or silicate gels as a practical adsorption medium, as was shown early in the investigations.

Foaming of ABS

The minimum concentration of ABS which will produce obvious foaming tendencies in aerated or vigorously agitated waters varies widely. The development of foam is dependent on the mineral organic content of the water and upon its pH and temperature. Preliminary explorations in search of an experimental "standard" water indicated that the addition of low concentrations of magnesium ion—in the range 5–7 ppm Mg^{++}—would yield test waters which produce in-

creased foaming at levels of 0.5 ppm ABS. Distilled-water samples vary widely in their tendency to foam with added ABS; natural river and pond waters are generally more sensitive than distilled water.

The foaming tendencies observed with a few parts per million of ABS should be differentiated from stable voluminous foams which are sometimes observed in sewage treatment plants. At minimum levels of ABS, a mere tendency to bubble is observed. At ABS concentrations of less than 5 ppm—in distilled water, for example—only a slight and very short-lived, unstable foam is produced.

TABLE 1

ABS Removal Efficiency of Activated Carbons

Activated Carbon Applied *ppm*	Grade Applied*	Initial ABS Concentration—*ppm*			
		0.20	0.50	1.00	2.00
		ABS Remaining in Solution—*ppm*			
1	S	0.18			
2	S	0.16	0.38		
	WT	0.19	0.42		
5	S	0.12	0.32	0.77	
	WT	0.17	0.38	0.96	
10	S	0.06	0.22	0.26	1.85
	WT	0.15	0.29	0.93	1.90
20	S	0.03	0.11	0.16	1.19
	WT	0.08	0.25	0.84	1.80
50	S		0.05	0.08	0.45
	WT	0.02	0.10	0.43	1.16
100	S			0.06	0.16
	WT		0.04	0.09	0.48
200	S				0.09
	WT			0.04	0.15

* S, selected grade; WT, water treatment grade.

TABLE 2

ABS Removal Efficiencies of Fifteen Grades of Activated Carbon*

Grade of Activated Carbon	Activated Carbon Applied—*ppm*		
	20	50	100
	ABS in Supernatant—*ppm*		
1	0.04	0.008	0.005
2	0.10	0.030	0.010
3	0.04	0.010	0.004
4	0.12	0.050	0.008
5	0.13	0.060	0.014
6	0.03	0.008	0.005
7	0.06	0.020	0.014
8	0.08	0.014	0.007
9	0.10	0.025	0.017
10	0.16	0.120	0.007
11	0.10	0.036	0.014
12	0.02	0.004	0.004
13	0.10	0.030	0.008
14	0.06	0.020	0.014
15	0.11	0.050	0.011

* 0.2-ppm ABS solution prepared in saturated magnesium–calcium carbonate system.

Adsorption Efficiencies of Activated Carbons

The practice and economics of removing ABS from treated waters with activated carbon will depend upon the degree of removal required. It will also depend upon the initial concentrations of ABS in the raw water. This is readily demonstrated by the data of Table 1.

In Table 1 the performances of two types of activated carbon are compared; that of an inexpensive water treatment grade of activated carbon widely used for taste and odor control, with good ABS-adsorbing capacity, and that of a selected grade of activated carbon with high ABS-absorbing capacity. The data in Table 1 represent the concentration of ABS remaining in water which initially contained 0.2–2.0 ppm ABS and which was

treated with activated carbon in concentrations varying from 1.0 ppm to 200 ppm.

It may be seen at once that the activated-carbon concentration range studied goes much higher than those commonly applied for taste and odor treatment. If the more efficient activated carbon is used, 5–10 ppm carbon must be applied to reduce 1.0 ppm ABS by 50 per cent, or 0.5 ppm, and about 40 ppm must be applied to reduce it by 90 per cent, or 0.1 ppm. The less efficient grade will require nearly 50 ppm activated carbon to bring 1.0 ppm ABS down to 0.5 ppm, and around 100 ppm to reduce the ABS by 90 per cent.

The choice of activated carbon for taste and odor removal is determined by overall economics. The concentrations required for drastic reduction of ABS may justify the search for specific activated carbons. Different grades of activated carbon vary widely in ability to adsorb ABS from dilute water solutions. The ABS-adsorbing capacities of fifteen different grades are compared in Table 2.

These values were experimentally determined by measuring the ABS residue in solutions initially containing 0.2 ppm ABS after exposure to 20, 50, and 100 ppm of the different activated carbons. Analytical variations at the low levels of ABS measured are to be anticipated, but it is possible to separate high-performance materials from less active materials. None of the activated carbons in these tests was chosen for ABS adsorption. The group represents a variety of activated carbons used in water treatment—some for taste and odor removal, others for removal of chlorine and chlorinated addition products.

TABLE 3

Effects of Low Concentrations of ABS on Phenol Adsorption by Activated Carbon

Type of Activated Carbon	Amt. Used *ppm*	No ABS Present	0.2 ppm ABS Present
		Phenol Remaining*—*ppb*	
A	8	17	17
	12	8.5	16
	18	6.5	6.5
B	8	43	55
	12	17	53
	18	3.8	22
C	8	37	25
	12	22	18
	18	9.7	8.6
D	8	45	46
	12	26	18
	18	19	14
E	8	24	21
	12	12	15
	18	6.0	13

* Initial concentration 100 ppb.

TABLE 4

Effect of Phenol on ABS Adsorption by Activated Carbon

Activated-Carbon Concentration *ppm*	No Phenol Added	0.5 ppm Phenol Added
	Final ABS Concentration*—*ppm*	
10	0.07	0.08
20	0.022	0.019
50	0.008	0.005

* 0.20 ppm ABS solution prepared in saturated magnesium–calcium carbonate system.

Effect of Phenol

There does not appear to be any useful relationship between the "phenol index" of activated carbon and its ability to take up ABS. The presence of ABS does not affect the phenol-

adsorbing capacity of the activated carbon, and the presence of phenol does not affect ABS adsorption. The data from experiments which demonstrate the practical specificity of activated carbons are given in Table 3.

In this group of experiments, water containing 100 ppb phenol was treated with three different concentrations of five different grades of activated carbon—8, 12, and 18 ppm. At each level, one member of a pair of samples received 0.2 ppm ABS. The phenol residue in the clarified waters shows no competitive effects of the ABS; the members of the pairs receiving ABS show comparable phenol residuals, within experimental error, to those containing no added ABS.

In another demonstration, 10, 20, and 50 ppm of a water treatment grade activated carbon were added to a series of paired samples, all containing 0.2 ppm ABS, one member of which received 0.5 ppm phenol. The ABS residuals in the clarified waters showed no displacement of ABS by added phenol (Table 4).

TABLE 5

Effect of Sewage on ABS Adsorption by Activated Carbon

Activated Carbon *ppm*	ABS Adsorbed—*per cent*	
	Carbon *A*	Carbon *B*
No Sewage		
10	<5	12
25	<5	18
50	9	39
100	44	72
10 per cent Sewage*		
10	<5	5
25	23	29
50	25	50
100	55	79
20 per cent Sewage*		
10	16	20
25	32	42
50	41	57
100	53	83
Polluted Creek Water		
10	<5	5
25	15	22
50	28	32
100	45	78

* Treated Imhoff tank effluent, 80 ppm BOD.

TABLE 6

Effect of pH on ABS Adsorption by Activated Carbon

pH		Final ABS Concentration* *ppm*
Initial	Adjusted	
7.0	8.0	0.083
7.1	6.5	0.078
7.1	5.0	0.072
7.9	8.0	0.0145
7.8	6.5	0.0120
7.9	5.0	0.0085

* Initial concentration 0.20 ppm.

Effect of Organic Matter

There is a second implication of practical importance in this demonstration of specific ABS-adsorbing capacities of activated carbons. Soluble and colloidal organic materials are present in all waters. When ABS is added to surface waters, a much higher concentration of assorted organic materials is added with the sewage that carries the detergent. Ultimately some of this is adsorbed on activated carbon used to treat downstream supplies. The authors' own brief explorations indicate that the adsorption of ABS is not competitively inhibited by domestic sewage.

If relatively high ABS concentrations (10 ppm) are mixed with 10 per

cent and 20 per cent settled domestic sewage, or with heavily polluted creek water, and samples are treated with varying concentrations of activated carbon, the ABS removed by the carbon on clarification is shown not to be affected by the amount of sewage or pollutional material present. This suggests a strong, specific adsorption of ABS by the activated carbon. The data from tests confirming this fact are presented in Table 5. When the experiment was repeated with two other grades of activated carbon, they varied in their specific affinities of ABS, but both showed the same indifference to added organic materials.

Effect of pH

The efficiency of ABS adsorption by activated carbon in acid waters is appreciably higher than in alkaline waters. This may be demonstrated by measuring ABS residuals in systems made increasingly acid with sulfuric acid and treated with activated carbon. The data in Table 6 show this. This experiment was an attempt to examine the effects of pH at carbon doses designed to remove approximately 50 per cent and 90 per cent, respectively, of an initial 0.2-ppm ABS loading.

Effect of Alum Treatment

Although tests have shown that the removal of ABS from waters containing 0.2–2.0 ppm ABS by alum coagulation is negligible at pH values above 5.0 (small concentrations are dragged down by silt and colloids trapped within the alum floc), it does appear that alum clarification of water treated with activated carbon appreciably increases the efficiency of adsorption by carbon. This is demonstrated in Table 7, which presents data from part of a group of experiments to examine the effects of combined high ABS and high activated-carbon concentrations upon alum flocculation. It is possible that the increased adsorption of ABS on the activated carbon in this case was related to the lower pH values of systems receiving 34 and 68 ppm alum, but the increase in efficiency is somewhat greater than experience with acidified systems indicates.

TABLE 7

*Effect of Alum Flocculation on ABS Adsorption by Activated Carbon**

Alum Added *ppm*	pH After Floc Formation	ABS Adsorbed†	
		ppm	*per cent*
0	7.1	0.49	24.5
17	6.5	0.52	26.0
34	6.3	0.60	30.0
68	5.3	0.88	44.0
0	5.1	0.74	37.0

* 50 ppm.
† Initial concentration 2 ppm.

TABLE 8

Adsorption of ABS on Various Fine Inorganic Solids in Suspension

Material Added	Concentration in Suspension *ppm*	ABS Adsorbed* *per cent*	Ratio of Added Material to Adsorbed ABS
Silica gel	1,000	13.0	1,540
Diatomaceous earth	1,000	13.4	1,495
Talc	1,000	54.3	369
China clay	800	19.2	835
Calcium carbonate	250	12.8	390

* Initial concentration 5.0 ppm.

TABLE 9

Adsorption of ABS on Silt Suspensions

Initial ABS Concentration *ppm*	Silt Load (Dry Weight) After 5 days' Standing *ppm*		ABS Adsorbed *per cent*		Ratio of Silt to Adsorbed ABS	
	Settled	Unsettled	Settled	Unsettled	Settled	Unsettled
0.5	9,350	12.5	13.3	2.0	218,500	1,250
0.5	1,840	17.5	13.4	2.8	42,800	1,250
0.5	256	2.5	9.3	2.7	7,320	205
0.5	4,920	17.5	17.5	2.6	43,600	1,350
0.5	308	12.5	27.5	21.0	2,240	119
0.5	1,460	120.0	4.5	1.5	328,000	16,000
2.5		17.5		13.4		52
2.5		5.0		6.0		33
2.5		5.0		7.3		27

All phases of water treatment that remove solids from the water also remove ABS adsorbed on the solids. The removal with clarification may be appreciable. Most of the solids which may be added to water during controlled water treatment have a limited specific absorption capacity for ABS. Data on the comparative adsorption of ABS on silica gel, diatomaceous earth, talc, china clay, and precipitated calcium carbonate from lime softening are given in Table 8.

Effect of Clarification

Some of the natural waterborne silt materials have relatively high ABS adsorption capacities, and there may be conditions under which clarification of silty water appreciably reduces the ABS concentration of the finished water. Data on the performance of a series of nine samples of local silted waters, enriched with 0.5 ppm ABS and clarified by filtration through membrane filters, are presented in Table 9. The ABS adsorption capacity of the more stable silt fraction—that which remained in suspension for 7 days—was compared with the heavier silt materials that settled out in less than 7 days. The performances were as anticipated; the finer materials in all cases adsorbed 20–100 times as much ABS per unit dry silt as the heavier fractions. Three samples of stable silt showed remarkably high adsorption capacities, approaching those of activated carbons.

These observations are essentially exploratory, however; much more carefully controlled work is required for a proper evaluation of the role of silt in the removal of ABS from waste-bearing waters. It is nevertheless obvious that there are additional benefits to be had through improved clarification of treated waters.

Plastic Films for Water Storage

C. W. Lauritzen

A contribution to the Journal by C. W. Lauritzen, Project Supervisor, Western Soil & Water Management Research Branch, Soil & Water Conservation Research Div., Agricultural Research Service, US Dept. of Agriculture, Logan, Utah.

POLYETHYLENE and vinyl films are gaining wide acceptance as linings for irrigation canals and reservoirs, because they control seepage effectively and are low in cost. Among the prospective applications of these plastic films, low-cost storage would seem to be the most promising. The properties of these films, as well as the principles of their construction and examples of their use for water storage, are discussed below.

Film Characteristics

Thin films of both vinyl and polyethylene are structurally weak, but if kept intact they are almost completely watertight. Polyethylene films are less expensive and black films have better aging properties than vinyl, especially when exposed. Vinyl, on the other hand, is much more resistant to impact damage and is readily seamed and patched with a solvent cement, whereas polyethylene can be joined or patched satisfactorily only by heat sealing.[1] It should be observed, however, that in recent years better polyethylene adhesives have been developed. This, together with the better aging and lower cost of polyethylene films, makes them attractive and deserving of more serious consideration for water barriers of all types.

Earth Reservoir Linings

Because of the weakness of thin plastic films, they must be protected from mechanical damage if they are to be serviceable. It is for this reason that buried film linings are recommended for reservoirs. If the reservoir is fenced and kept filled or partially filled, exposed polyethylene film linings will give reasonably good service. Vinyl linings, on the other hand, tend to deteriorate in the region above the water level after 2–3 years and covering is necessary. Only the side slopes need to be covered, however, if provision is made for 1 ft of water or more to re-

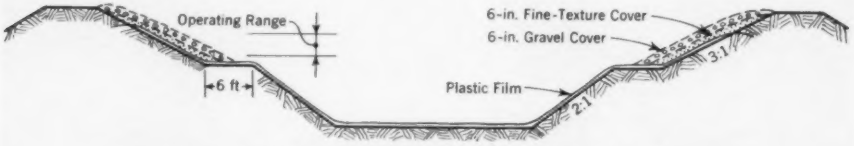

Fig. 1. Broken-Slope Design for Plastic-Film-Lined Reservoirs

This design is practical only where the water level is regulated so as not to decline below the break in slope.

main over the bottom of the reservoir at all times. This depth is sufficient to protect the film from degradation due to exposure, and usually there is sufficient sediment in irrigation water to provide a protective covering after the first season. This minimum depth of water can be maintained if the outlet intake is located 1 ft above the bottom.

of the reservoir. If the operation of the reservoir is such that the water level is regulated between definite limits, a broken-slope profile can be used to avoid the necessity of covering the entire slope (Fig. 1).

Certain plants penetrate both vinyl and polyethylene film of a thickness of 8 mil or thinner. Film of this thickness has been used for lining most

Fig. 2. Positioning of Vinyl Film in Reservoir

Shown is the installation of the first 61×300-ft section. Sections were bonded at the edges with a solvent cement.

In order for the earth cover material to remain in place, the slope of the sides should not exceed 3:1. If the slope is steeper than this, the cover material may slide, exposing the film. The cover can be replaced and built up to a slope of 3:1 and the material will stabilize. The result, of course, will be a reduction in the storage capacity

reservoirs in the past. The susceptibility of film to penetration by growing plants has been demonstrated by controlled tests as well as field observations.[2] For this reason, it is desirable to sterilize the subgrade, particularly the side slopes, in areas where nut grass, Johnson grass, quack grass, and other plants having a high penetrating

power are present. Several good chemicals for this purpose are commercially available.

If the subgrade is very coarse in texture or contains sharp stones, it should be dressed with a fine-textured cushion material. This is particularly true if polyethylene film is to be used. The standard recommended for the cover material is 6 in. of fine-textured material followed by 6 in. of gravel. The fine-textured material is used as a cushion for the gravel to prevent erosion and eventual exposure of the film on the side slopes. There is some question about the necessity for cover of this thickness, particularly if vinyl film is used. An 8-mil vinyl film was only very slightly damaged by gravel dropped from a height of 20 ft. Polyethylene, on the other hand, was badly damaged from a drop of 10 ft. There is evidence that it would be more economical to use a heavier-gage film and dispense with some of the cover.

During 1960, four reservoirs just north of Salt Lake City were lined as a part of a pressure irrigation system. The reservoirs had capacities of 1, 5, 9, and 20 acre-ft. The first three of these were lined under one contract at a cost of $2.04 per square yard. The cost of the vinyl film in place was 47½ cents per square yard. The remainder of the cost was for the cover. The cost of covering seems excessive. A part of this cost at least can be explained by the fact that the cover material was hauled some distance.

Fig. 3. Placement of Earth Cover and Gravel Topping Over Vinyl Film

Fine-textured earth cover was placed over the entire invert of the reservoir, and gravel was then placed on the slopes.

The largest of the reservoirs was lined at a cost of $1.41 per square yard. Of this, 56 cents was for the

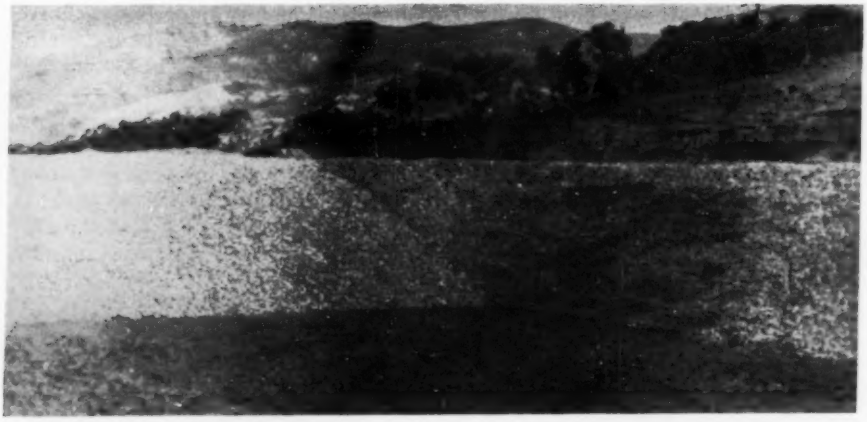

Fig. 4. Vinyl-Film-Lined Reservoir With Cover Material in Place

Shown is the reservoir after completion of the operations illustrated in Fig. 3.

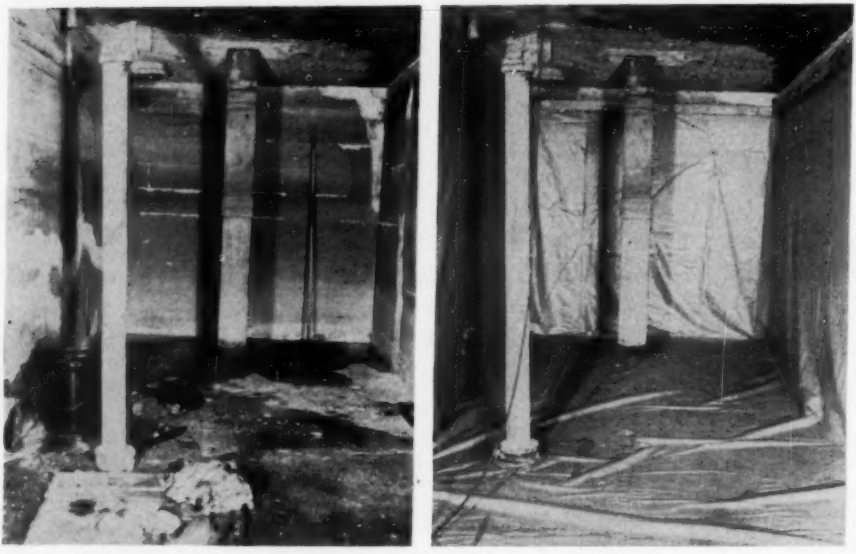

Fig. 5. Concrete Reservoir Before and After Lining With Vinyl

The 20-mil vinyl was rolled around wooden strips anchored at the top of the reservoir with ramset nails.

vinyl film in place and the rest for cover. The overall cost of the reservoir, including structures, was $3.30 per square yard. The lining in this latter reservoir was installed in four pieces 61×380 ft and required 380×3 ft, or 1,040 ft, of field seaming. The seams consisted of a 6-in. overlap bonded with a solvent cement. The cover consisted of 6 in. of a fine-textured earth over the entire liner topped with 6 in. of gravel on side slopes. Figure 2 shows a 61×300-ft sheet of 8-mil vinyl which has just been spread into position. This sheet came in one piece accordion-folded on a pallet. Less than 30 min was required to unpack and position the sheet as shown. The section liner was looped out of the back of a truck along the top of the reservoir. In this position it was still accordion folded in one direction. The positioning was completed by several men taking hold of the leading edge at intervals along the sheet and walking forward. To anchor the lining it was buried in the trench along the top. The next strip, also 61 ft wide, was looped out of a truck along the edge of the first piece and the edges were bonded with a solvent cement. The fine-textured cover and gravel topping are shown being placed in Fig. 3, and the completed lining in Fig. 4.

Linings of the type described above have been in service 6 years and have provided complete seepage control. Just how long such linings will last can only be estimated, but it would appear from accelerated tests and from the condition of the films that they will last for many years.

Repairs to Old Reservoirs

Another application for plastic film is in the repair of leaky reservoirs or tanks constructed of concrete, metal, or lumber. The lining can be installed in the same manner as it can in earth reservoirs. If the side slopes are too steep for earth cover to remain in place, some other type of protection must be afforded on the area above the operating water level, unless the reservoir is covered, as some domestic-supply reservoirs are. In vertical-wall structures it may not be feasible to bring the film up over the edge and bury it. The film can be attached to the wall by rolling the top edge of the film on a piece of lumber and fastening it to the top of the structure.

A concrete reservoir used by Logan City in the cooling system of its diesel electric plant became so extensively cracked that it would not hold water overnight. Many repair measures were tried without success. In April 1955 a 20-mil vinyl lining was installed. The lining completely stopped the leakage. After more than 5 years the lining is as effective as when it was installed and, from all appearances, will give many more years of service. The liner was fabricated as an open bag and the top edges were attached to the top of the inside wall with a nailing strip on which the film was wrapped. Photographs of this reservoir before and after lining are shown in Fig. 5.

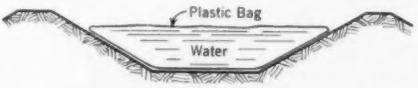

Fig. 6. Plastic Bag in Reservoir for Control of Seepage, Evaporation, and Algae

The bag serves only as a seal; the reservoir walls take all the stresses.

Other Uses

It is the author's understanding that there is a problem in some areas of recontamination of water stored for domestic purposes. The use of a plastic bag or plastic reservoir cover might well be an effective and economical means of controlling contamination and evaporation.[3] Some work has been done by the author on the use of film coverings to control aquatic growth and evaporation from small reservoirs. When the covers were kept in place on the surface, a black film effectively controlled both aquatic growth and evaporation. It was difficult to keep the cover in place, however. A better approach would appear to be the use of a large bag or liner with an attached cover. The bag would not have to be shaped to fit the reservoir; it could consist simply of two sheets of film welded together on the edges with an inlet and outlet tube. If an opaque film is used, algae will not develop as they do in open reservoirs. The film bag should be large enough that when filled with water the film would not be subject to stresses and the stress would be taken by the reservoir invert. A closed plastic bag supported and confined by the reservoir invert is illustrated in Fig. 6. If the volume to be stored is small, such as might be required for emergency home use, a reinforced bag might be used, eliminating the need for a confining structure.

Conclusion

Many uses have been found for plastics. A recent development has been the use of film liners for ponds, small reservoirs, and tanks. Although this development has been directed toward irrigation storage, the liners have also been used for the storage of water for domestic use and for industry. The good performance of film linings for such applications would indicate their potential applicability in water works structures.

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Status of Fluoridation in Puerto Rico

Ramon M. Guzman

A contribution to the Journal by Ramon M. Guzman, San. Engr., Puerto Rico Aqueduct & Sewer Authority, San Juan, P.R.

SINCE fluoride salts were first added to the water supply of Grand Rapids, Mich., on Jan. 25, 1945,[1] an increasing number of communities has adopted this program as a prophylactic measure to reduce the incidence of tooth decay among the young population. The application of fluorides to water supplies has not increased in direct proportion to the growth in population, however. During the last 3 years, the population of the United States has increased by 7,500,000, but the number of inhabitants receiving fluoridated water increased by only 4,200,000. Thus the proportion of the population not receiving the benefits of this remarkable health measure is actually increasing.[2] The opposition from various minority groups has succeeded in preventing the adoption of fluoridation in many communities, but apparently the opposition has had little effect on communities where fluoridation is already practiced. In 1958, only two United States water systems, serving a total of 15,130 people, discontinued fluoridation.[3]

Fluoridation in Puerto Rico

In Puerto Rico, progress in water fluoridation has been positive in all respects. The Department of Health of Puerto Rico, conscious of the benefits that a fluoridation program would offer the Commonwealth population, began investigating the possibility of adopting such a program in 1951. USPHS was asked to cooperate with the Insular Department of Health by providing technical advice, and in 1952 the Puerto Rico Legislature approved a project to add artificial fluorides to the island's water supplies.

The Puerto Rico Aqueduct & Sewer Authority, a self-supporting agency of the Commonwealth of Puerto Rico,[4] was charged with the responsibility for implementing the program, as well as the operational procedures necessary to keep it running.

After the approval of the public law authorizing the fluoridation of water supplies in Puerto Rico, the legislature appropriated funds for the purchase of the equipment. Funds were also provided for the operation and maintenance of the program. It should be pointed out that fluoridation is the only water treatment phase financed by the government, as funds for all other aspects of water treatment are derived from revenues. A total of $250,000 was appropriated to start the fluoridation program. For the fiscal year 1960 the fluoridation program cost a total of $641,000. It is expected that for the next year $75,000 will be spent on the fluoridation program, not including depreciation of equipment and amortization of funds. This is equivalent to 5.2 cents per capita per year.

Initiation of Program

Inasmuch as water consumption in metropolitan San Juan accounted for 53 per cent (year 1953) of the authority's total water production, this area was the logical choice to start the program. At that time the production of the San Juan water plant was 30.7 mgd; the combined output of the Loíza and Guaynabo filtration plants. The application of artificial fluorides to the Commonwealth water supplies was started on Nov. 17, 1953, thus making it the first program of its kind in Puerto Rico, as well as in all of Latin America.

Rural Areas

Puerto Rico has a population of 2,314,000 and an area of 3,450 sq mi, for a population density of 673 per sq mi. A large number of its inhabitants live in the rural areas. To serve this population, the distribution systems of the urban water supplies are extended so as to reach the rural areas. Of course, this is not always economically feasible; if not, rural water supplies are developed. Because of the low water consumption in these systems and the small population served by each of them, it is not now considered practical to fluoridate the water they serve. At the end of the fiscal year 1960, there were 111 rural systems serving 126,000 families with bacteriologically safe water.

TABLE 1

Summary of Fluoridation Practice in Puerto Rico, 1960

Item	Metropolitan San Juan	Other Municipalities	Total
No. of communities	11	58	69
Population served—*1,000's*	652	769	1,449
Water produced—*bil gal*	20.27	13.68	33.95
Fluorides applied			
amount—*lb*	188,684*	184,156†	372,840
concentration—*lb/mil gal*	9.3	13.5	
Available F in finished water—*ppm*	0.67	0.71	

* Sodium fluoride.
† Sodium silicofluoride.

Extension of Program

The fluoridation program was extended rapidly to other communities equipped with water filtration plants. In 1954 the program was started successfully in nine other communities, and at the end of the fiscal year 1960 fluoride salts were being added to 62 installations serving 69 Puerto Rican communities. The eight communities not yet receiving such service are scheduled to receive fluoridated water from new plants, two of which will be in operation early in 1961. Data on the status of fluoridation at the end of 1960 are given in Table 1 and Fig. 1.

It is expected, however, that once all the urban systems produce fluoridated water, the percentage of the population receiving it will continuously increase, as the constant rise in living standards being experienced has created an exodus from the rural areas to the urban centers and their suburbs.

Population Served

At the end of 1956, 1,487 communities with a population of 31,416,000

were serving fluoridated water in the United States.[5] At present, water sources for 3,703 United States communities with a total population of 42,000,000 contain natural fluorides or are artificially fluoridated. One out of three United States residents using central water supplies receives fluoridated water, or 23 per cent of the total population. By way of contrast, in Puerto Rico, 93 per cent of the population using water processed through filtration plants receives the benefits of fluoridation, which is equivalent to 63 per cent of the total population. In Puerto Rico a higher percentage of the population is served fluoridated water than in any other country in the world. Figure 1 shows the proportion of water production in the Commonwealth to fluoridated water.

Fluoride Feeders

Water filtration plants serving Puerto Rico operate at various capacities, ranging from 0.25 to 40 mgd. A variety of fluoride feeders was therefore chosen to meet each specific requirement.

It has been considered practical to use solution feeders at plants of limited capacity. Although various federal agencies recommend the use of solution feeders when the plant capacity is less than 2.0 mgd, in Puerto Rico the use of solution feeders for the injection of sodium fluoride has resulted in poor control of the ultimate fluoride-ion concentration. Because of these results, solution feeders were eliminated in favor of dry feeders, even at water plants with productions as low as 250,000 gpd. In small deep well systems, solution feeders are synchronized with the pump motor and the fluoride solution is injected as water is drawn from the wells.

Chemicals

Chemicals being used to supply the fluoride ion are sodium silicofluoride and sodium fluoride. The former is used in the water plants serving metropolitan San Juan; the latter is used in all other systems. Sodium fluoride was at first used at the San Juan plants but was later abandoned in favor of the silicofluoride for economic reasons. Sodium fluoride is used in other island

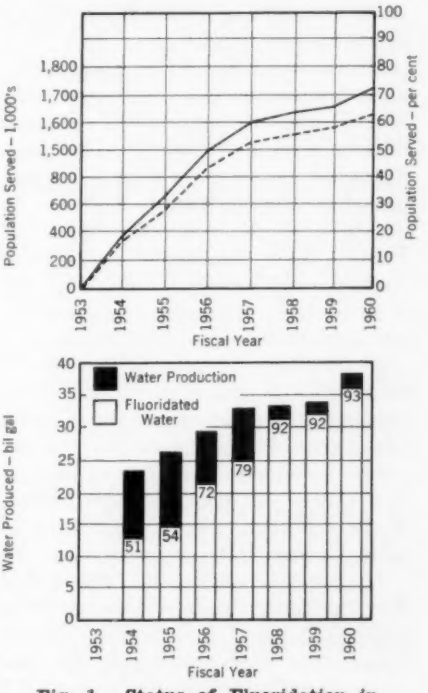

Fig. 1. Status of Fluoridation in Puerto Rico

The lower graph gives the total water production for the years 1953–60 and the percentage of that production that was fluoridated. In the upper graph, the solid curve represents the total population receiving fluoridated water; the dashed curve represents the percentage of the population receiving fluoridated water.

communities because of a minimum effort needed to obtain water with optimum fluoride residuals. Specific operations, to a great extent, predict what chemical should be selected.

Fluoride-Ion Concentrations

Through correspondence with USPHS in 1952 it was decided that the optimum fluoride-ion concentration for the waters of Puerto Rico should be 0.8 ppm. More recent research work correlating the fluid intake among children with prevailing temperatures indicates that for mean maximum temperatures in the range 79.3–90.5°F the recommended optimum fluoride level is 0.7 ppm.[6] The mean maximum temperature over a 30-year period in Puerto Rico is 85.9°F. In the light of these investigations, the Puerto Rico Aqueduct and Sewer Authority is presently studying the desirability of reducing the fluoride-ion level to 0.7 ppm.

A fluoride-ion concentration in the range 0.7–0.9 ppm is now maintained, although there occur at times deviations from this range, as is to be expected. Seldom does the concentration exceed 1.0 ppm, however, which is the optimum dosage considered in most United States communities.

It should be noticed in Table 1 that the available fluoride injected was about 0.68 ppm. The average natural fluoride-ion concentration in the raw waters entering the plants is about 0.1 ppm, thus increasing the fluoride concentration to 0.80 ppm in finished water.

Control of the Fluoride Level

Most water systems have some difficulties in maintaining a constant fluoride level. The maintenance of a given fluoride-ion concentration in a public water supply is not always an easy task, although it may seem theoretically simple. The precise control of any given dosage of chemicals added to a water supply is always difficult, unless the water plant is equipped with special control instruments.

Experience in Puerto Rico has shown that control of the fluoride-ion concentration is most difficult in those systems that derive the water from ground sources. This water is served directly after chlorination and fluoridation. In order to prevent too high a dosage in the water supplied by these systems, the injection of fluoride is set low at times because an underdose is preferable to an overdose.

TABLE 2

Reduction in DMF Index During 1953–58

Age *years*	Reduction in DMF Index *per cent*
6	65.9
7	49.1
8	42.7
9	43.9
10	44.3
11	42.4
12	43.1
13	27.2
14	31.7
15	31.0

Program Supervision

Puerto Rico is arbitrarily divided into six water districts, each one under the supervision of a sanitary engineer in charge of water treatment. Continuous supervision assures the adequacy and safety of the water served. The Operations Division of the Aqueduct & Sewer Authority receives water samples at least once a week from all systems. Fluoride-ion determinations are performed on each sample at a central laboratory in San Juan. The

Commonwealth Department of Health as well collects water samples as a double-check. Last but not least, fluoridation concentrations are determined daily by individual water plant operators by the Megregian-Maier method, as described in *Standard Methods.*[7]

Results of Fluoridation

Plans for the implementation of Puerto Rico's fluoridation program included a prefluoridation survey similar to those made in Grand Rapids and Muskegon, Mich. and in Newburgh and Kingston, N.Y.[6] In 1951, a team of dentists examined 974 children in the 6–15-year age group. The survey indicated a DMF index * of 1.23. In 1958 a total of 1,892 children aged 7–15 who had used fluoridated water during the 5-year period were examined. Among the children aged 6 years, a DMF index of 0.42 was established, thus indicating a reduction of 65.9 per cent[9] in tooth decay in 5 years, but this figure was lower for the children aged 7–15. Table 2 gives the results of the survey in the different age groups. Further evaluation of results is scheduled for 1963, which will be 10 years after the program was implemented.

It is possible that not all the reduction of dental caries in Puerto Rico's young population is due to fluoridation. Improved dietary habits of the Puerto Rican population may have helped as well. Meats, proteins and other nutritious foods are replacing many of the stomach-filling carbohydrates preferred in the past. Better health habits among the population and educational programs on oral hygiene have also been of evident help.

* Decayed, missing, or filled permanent teeth per child.

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Adjustable-Speed Pumps for Utilities

Adam W. Kubik and Kenneth H. McEwan

A contribution to the Journal by Adam W. Kubik, Cons. Engr., O'Brien & Gere, Syracuse, N.Y., and Kenneth H. McEwan, Div. Salesman, D-C Specialty Drive Sec., Westinghouse Electric Corp., Buffalo, N.Y.

PERFORMANCE characteristics of adjustable-speed centrifugal pumps show that this type of unit is very well suited to systems for which pipe friction is a predominant factor to be considered. The selection of the drive for an adjustable-speed pump depends primarily on the accuracy and dependability of control that may be required, although the horsepower rating and the characteristics of the available power supply may also be factors affecting selection.

Where adjustable-speed pumps are used in place of constant-speed pumps, fewer units are generally required. Also, where an open suction well is required, the close control of the pumping rate—made possible with the speed control—permits a considerable reduction in the suction well capacity. Savings in the overall project cost, due to the use of fewer units and a smaller structure, frequently more than offset the initial higher cost of variable-speed drives and their control. In addition, very substantial savings in power costs are often possible, owing to improved operating efficiency of a modern adjustable-speed unit as compared to a constant-speed unit.

The pumping station designer must often provide for widely varying operating conditions, both in the rate of flow to be handled and in the total head to be developed. These requirements are dictated by the daily and seasonal variations in the system rate of flow or water consumption and by the effect of pipeline friction. In addition, provision must usually be made for the handling of increased future capacity.

Development of the system head-capacity curve is always the starting point in the design of a pumping station. This curve is composed of the static head, which is the difference between the suction and discharge gradients, and the pipe losses. For most systems, the static head remains relatively constant, regardless of the rate of flow. Pipe losses comprise losses through valves and fittings and from line friction, and are approximately proportional to the square of the rate of flow. The sum of the static lift and the various losses is commonly known as the total head.

System Curves

The system head-capacity curves (Fig. 1) for most water pumping stations may be classified as three types:

1. Type *A* is a curve in which the static head component is a considerable portion of total head at the design capacity—typical for an outlying sewage lift station.

2. Type *B* is a curve typical for an installation with a long transmission main for which pipe friction is the

predominant factor and static head is negligible.

3. Type *C* is a curve typical for a pressure booster station on a water distribution system in which water may flow by gravity past the station

Pump and Drive Selection

When pumping equipment is selected, proper attention must be given to first cost, annual operating cost, mechanical layout of equipment and piping, availability of power, and con-

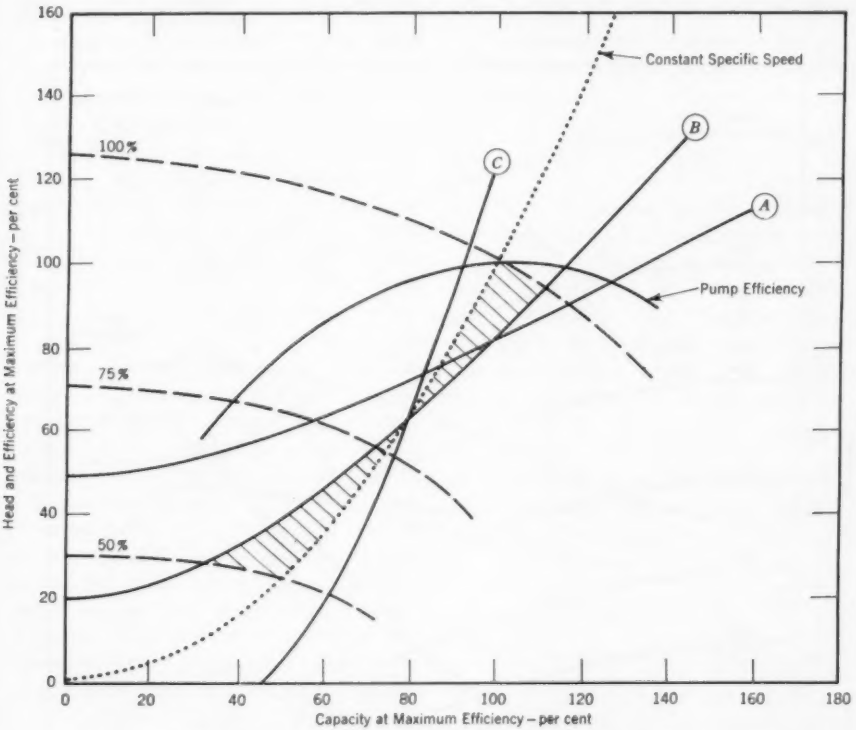

Fig. 1. Application of Adjustable-Speed Pumps

A curve designated by a percentage is a head-capacity curve at the indicated percentage of pump speed (measured in revolutions per minute). Circled letters pertain to types of system head-capacity curves: A, *system with approximately equal components of static head and pipe friction;* B, *predominantly frictional system; and* C, *system with negative static lift. The shaded area between the curve of constant specific speed and Curve* B *represents an efficient balancing of pump and system characteristics.*

during the average or below-average demand periods. At maximum periods, pumping against high heads is required. This type of system is characterized by very steep head-capacity.

trol problems. Selection of pump drives depends on control, characteristics of the available power supply, and economic factors. The number of pumps to be used in a given station

is generally established by the hydraulic considerations alone.

On most Type *A* systems, parallel operation of two or more units equipped with constant-speed drives usually gives good efficiency and controllability. Depending on the total range of flow and on the actual relationship between the static lift and the total head at maximum flow, two, three, or more pumps may be selected. If constant-speed pumps are used, capacity control can be effected only by a proper selection of the start-stop cycles for individual units or by the throttling of the discharge line. The

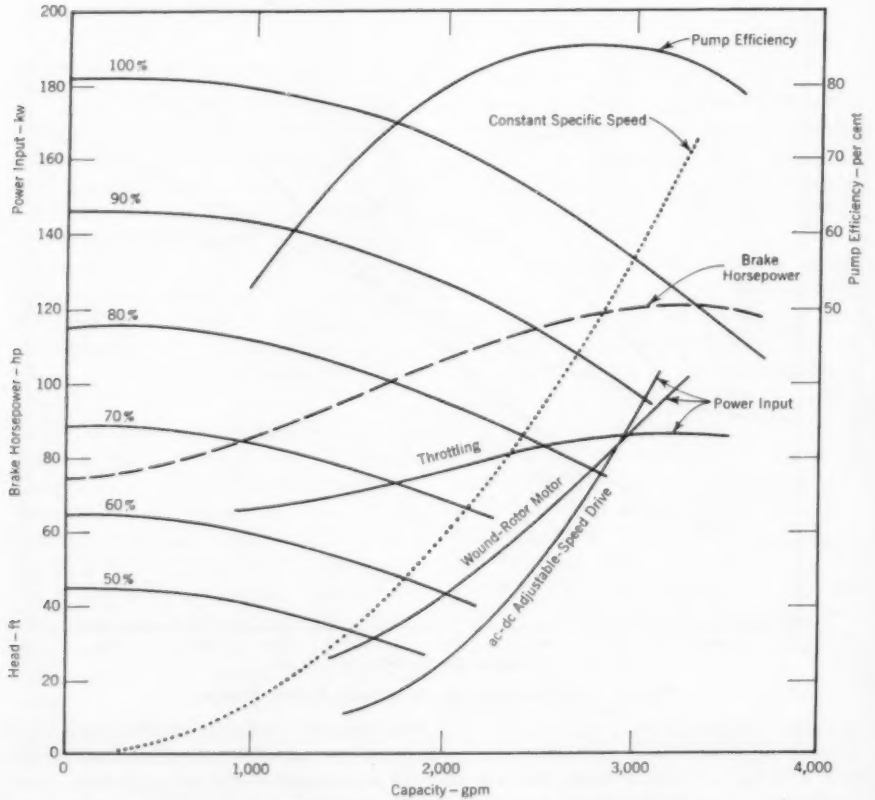

Fig. 2. Comparisons of Methods for Capacity Control

Shown are the results of capacity control by throttling at constant speed as opposed to control by variable speed, wound-rotor motor, and ac-dc adjustable-speed drives. A curve designated by a percentage is a head-capacity curve at the indicated percentage of pump speed. The dotted curve of constant specific speed closely approximates the median system curve at the Park Street Pumping Station in Syracuse, N.Y. The pump plotted is the pump actually used at the installation.

latter is the most uneconomical of the available methods to control pump capacity and is seldom used.

With almost universal distribution of a-c electric power, the "squirrel cage" induction motor is the simplest and often the lowest-cost device for a centrifugal-pump drive. It has the disadvantage, however, of "hard starting"—that is, of imposing a heavy starting-current inrush on the power supply system which often results in the additional cost of reduced-voltage starting equipment. Also, sudden large increases in pumpage, resulting from the starting of additional units with the high acceleration inherent in the induction motor, may cause harmful surging of the transmission mains.

In systems for which the higher initial cost of adjustable-speed pumping units can be economically balanced against simplified construction, owing to the use of fewer units or a smaller suction well, adjustable-speed pumping will give good results. The speed range required for most Type *A* systems will seldom exceed 25 per cent reduction below the rated pump speed. Multispeed pumps are not generally applicable to Type *A* systems, because the relatively large static-head component of the total head permits only a small speed reduction below the rated speed. For example, a 1,200/900-rpm pump would develop, at the speed of 900 rpm, a head that is only $\left(\frac{900}{1,200}\right)^2$, or about 56 per cent of the head at 1,200 rpm (Fig. 2).

Type *B* and Type *C* systems generally require adjustable-speed pumping because of the large increase in the total head that takes place as the pump capacity is increased. Where intermittent pumping is not possible or desirable, because of the control or other considerations, adjustable-speed pumping usually is the only feasible means of effecting capacity control.

Similarity relationships state that as the speed of a pump is decreased, the capacity is reduced; the head develops as the square of the speed; and the water horsepower develops as the cube of the speed. The equation for specific speed is:

$$N_s = \frac{NQ^{\frac{1}{2}}}{H^{\frac{3}{4}}} \quad \ldots\ldots\ldots\ldots (1)$$

in which N_s is the specific speed (dimensionless); N, speed of rotation (rpm); Q, capacity (gpm); and H, total head (ft). From the similarity relationships and from Eq 1, it follows that the locus of constant specific speed (as the rotative speed of a pump is varied) is a parabola of the form:

$$H = kQ^2 \quad \ldots\ldots\ldots\ldots (2)$$

in which k is a proportionality factor. Mathematically, the similarity relationships are expressed as:

$$Q = k_1 N \quad \ldots\ldots\ldots\ldots (3)$$

$$H = k_2 N^2 \quad \ldots\ldots\ldots\ldots (4)$$

$$\text{Water horsepower} = k_3 N^3 \quad \ldots\ldots (5)$$

When Eq 3 and Eq 4 are substituted in the expression for the specific speed (Eq 1), the right-hand side of Eq 1 becomes a constant. In Eq 2, the constant k is obtained by substituting the coordinates of any one point (Q_1, H_1) on the pump characteristic to obtain the equation of the parabola passing through that point.

As long as the specific speed remains constant, the pump operates under hydraulically similar conditions, and the hydraulic efficiency remains substantially constant. Type *B* system curves,

where only a minor static component exists, will be found to parallel closely, and over a relatively wide range of hydraulic conditions, some parabola of the family $H = kQ^2$. When an adjustable-speed pump is applied to a system, the selection should, therefore, be based largely on the consideration of proper matching of the system characteristic and of the constant specific-speed characteristic, if good operating efficiency over a wide range is to result.

Operation of a single pump gives the best results with adjustable speed. When several pumps must be operated at adjustable speed in parallel, all units should be identical. The method of speed control should be such that the units are effectively synchronized, so that they are hydraulically equivalent to one larger pump. When properly applied, a medium-sized centrifugal pump working into a steeply rising system curve will often exceed an average efficiency of 80 per cent over a flow range of 2:1 or more.

Adjustable-Speed Drives

Adjustable-speed drives powered by an a-c distribution system have been successfully used for driving centrifugal pumps. The excellent operating records being accumulated for the adjustable-speed pump installations indicate their increased use in years to come. Some of these drives are:

1. Wound-rotor induction motor
2. Synchronous or induction motor driving a magnetic or hydraulic slip coupling
3. Multispeed induction motor
4. Induction motor driving a mechanical variable-speed transmission
5. Brush-shift or Schrage motor.

Only the first two have gained a wide acceptance, perhaps with the wound-rotor motor being more frequently used in the past. Its advantages include ready availability, standardized and reliable controls, and low starting-current inrush. Speed control of the step increment type is generally used.

Adjustable-speed drives using magnetic or hydraulic couplings driven by a-c squirrel cage or synchronous mo-

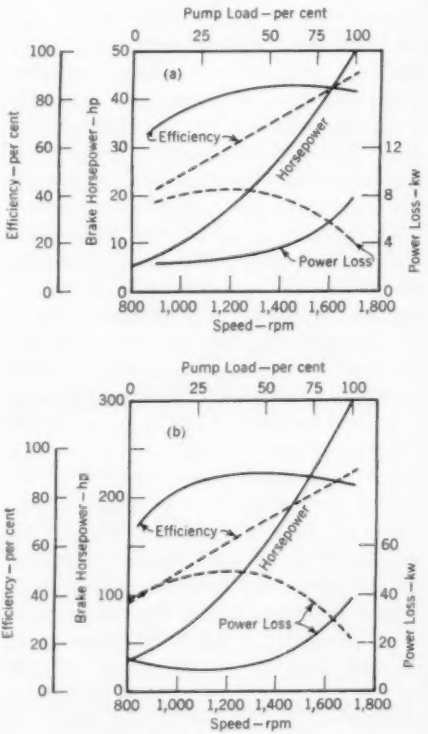

Fig. 3. Typical Operating Efficiencies of Various Pump Drives

Solid curves pertain to ac-dc adjustable-speed drives on a particular pump load: a 50-hp, 1,690/900-rpm drive in Part a*; a 300-hp, 1,700/800-rpm drive in Part* b*. Dashed curves pertain to a wound-rotor motor or eddy current clutch and squirrel cage motor on the same pump load.*

tors have the advantage of virtually stepless speed control, which is of particular importance where automatic and close speed control are required. But the overall efficiency, including the efficiency of the prime mover and drive, is usually slightly lower than that of the wound-rotor induction motor. In both the wound-rotor motor and the slip-type couplings, the speed reduction is effected by controlling the slip, which is the difference between the rotating speed of the primary magnetic field and of the output shaft of the drive.

The efficiency of all such drives is inversely proportional to the slip. The slip power, which at any speed is proportional to the slip speed and to the torque at that speed, appears as heat in the secondary resistors of the wound-rotor motor, as core and copper losses in the magnetic coupling, or as temperature rise of the fluid in the hydraulic coupling. In most instances, this is waste power converted to heat that must be removed. In larger installations, the removal of heat may require elaborate auxiliary systems.

The multispeed induction motor has a good efficiency at each speed and is relatively inexpensive. It will give good results where large changes in the capacity and head developed by the pump can be tolerated and where no starting limitations exist. Mechanical variable-speed transmissions, such as the variable-pitch pulley units, have found some field of application in the small horsepower ratings. The Schrage or brush-shift motor is quite complicated electrically and rather expensive. A popular drive in Europe, it has made a brief appearance in the United States, but is not now readily available. Its principal advantage is good efficiency over the full speed range.

ac-dc Adjustable-Speed Drive

A new entry into the field of centrifugal-pump adjustable-speed drives is an ac-dc adjustable-speed drive * that consists of an a-c wound-rotor component and a d-c shunt wound component mounted either on a common shaft or as two separate units. In the latter instance, the output shaft may be taken from either machine. Vertical as well as horizontal mounting arrangements are available.

The drive operates directly from standard a-c line power applied to the a-c wound-rotor component. As the speed of the drive is decreased, the a-c slip power, instead of being dissipated as heat, is rectified through silicon diodes to the armature of the d-c component to provide useful torque. The output capabilities of the drive are the sum of the torque outputs of the a-c and d-c components. Therefore, inherently the drive has a constant horsepower characteristic. Without special feedback systems, inherent speed regulation of the drive can be as low as 2 per cent in the higher horsepower ratings—that is, 200–500 hp. Generally, however, speed regulation is about 5–7.5 per cent over the entire operating range. The drive cannot exceed the synchronous speed of the wound-rotor component, as the units are normally directly coupled. When belted together, a mechanical overspeed device is always supplied to limit the speed of the d-c component in the event of a belt failure.

Where adjustable speed pumps are used, operation at full speed is generally required to meet only the maximum conditions, which occur infrequently. Usual operation is at a more

* RectiFlow drive, made by Westinghouse Electric Corp., Buffalo, N.Y.

or less reduced speed, and good efficiency under these conditions results in a considerable saving in the operating costs.

The efficiency of the ac-dc adjustable-speed drive is extremely high over the operating range, and the power factor is only about 5 per cent below that of an a-c squirrel cage motor of the same rating. Performance curves (Fig. 3) show typical operating efficiencies of an ac-dc adjustable-speed drive in comparison to efficiencies of wound-rotor and eddy-current drives.

Speed control of the drive is infinitely adjustable over its speed range through the variation of field current to the field of the d-c component. The control voltage for the field circuit is obtained by rectifying stepped-down a-c line voltage. In short, the ac-dc adjustable-speed drive has the operating characteristics and control flexibility of a standard d-c motor, but it can operate directly from an a-c power source. Starting current of the drive can be easily limited to 100–150 per cent of full load current. Normally, only one step of starting is required.

For automatic speed control, the field control rheostat of the ac-dc adjustable-speed drive can readily be equipped with an electric-motor drive or with a pneumatic operator, not unlike the actuator of a control valve. Pneumatic control can be arranged to

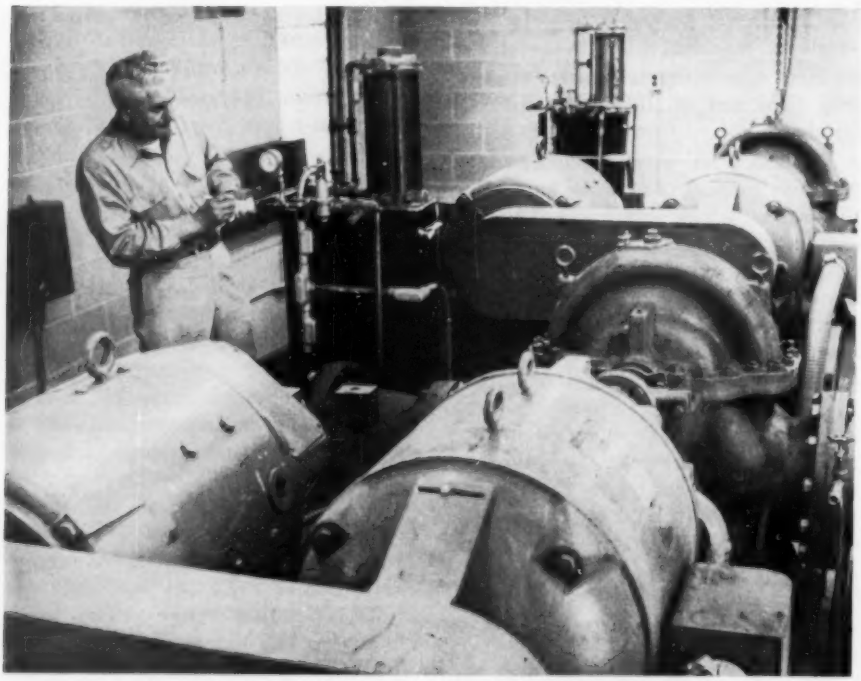

Fig. 4. Adjustable-Speed Drives at Park Street Pumping Station

Shown are the ac-dc adjustable-speed drives and their respective centrifugal pumps.

suit most control requirements. Proportional control action is readily achieved, with or without the reset action. Where very fast response is desired, a rate action may also be added. The drive itself has a very rapid inherent response to speed changes required by the system.

The effective output speed of the ac-dc adjustable-speed drive is a function of the division of the load between the a-c and d-c components, and the drive is inherently constant horsepower. Thus, it is possible to design the complete drive so that the full load on both components is approached at speeds other than the synchronous speed of the a-c machine. This feature permits considerable additional freedom in the selection of a pump for a given set of conditions.

Most pump manufacturers carry a line of pumps designed for 60-cycle motor speeds. Often, difficulty exists in selecting the pump for a set of conditions that are intermediate to the conditions optimum for a given pump operating at, say, 1,200 rpm and 900 rpm. Operation at about 1,000 rpm may result in an ideal application for a given pump. Where an ac-dc adjustable-speed drive is applicable to the job at hand, this additional freedom in pump speed selection may result in the use of a pump that is better, and often cheaper, for a given set of conditions than an alternate pump that operates at a standard 60-cycle speed.

Installation of Drives

The first existing municipal installation of centrifugal-pump ac-dc adjustable-speed drives is in a water pumping station owned and operated by the Onondaga County Water Authority at Syracuse, N.Y. The station, known as the Park Street Pumping Station, comprises two 125-hp units (Fig. 4), each rated as 3,000 gpm at 135 ft at the future maximum speed of 1,500 rpm. The present speed is limited in the control circuit to 1,420 rpm, which is equivalent to a pump

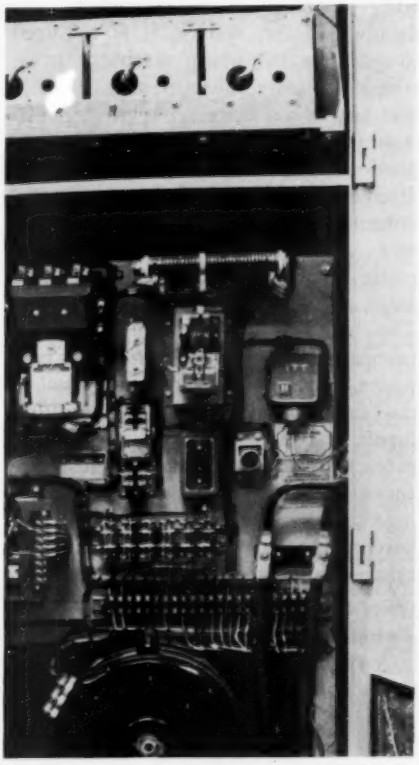

Fig. 5. Basic Control for ac-dc Adjustable-Speed Drive

The pneumatic actuator is behind, and connected directly to, the ac-dc drive speed control rheostat shown at the bottom of the illustration. Silicon slip rectifiers at the top of the illustration convert a-c slip power to d-c power, to provide d-c armature power. Silicon field rectifiers above the rheostat provide field excitation to the d-c unit.

capacity of 2,800 gpm at the total head of 125 ft.

The pumps in this installation are subject to a highly variable suction pressure, which is due to the hydraulic conditions in the water transmission mains. The station supplies water to a group of elevated tanks in an outlying district. For satisfactory hydraulics of the system, it is required to maintain the pump discharge pressure reasonably constant, regardless of the suction pressure.

A pneumatic controller * with proportional and reset action regulates the discharge pressure by applying control output pressure to a pneumatic actuator. The actuator is connected directly to the field rheostat of each ac-dc adjustable-speed drive (Fig. 5). The starting and stopping of the pumps is accomplished through a telemeter circuit as a function of level in the remote elevated tanks. Requirements of the system are such that the rate of flow through the station is reasonably stable, varying between 2.5 mgd and 4 mgd, depending on the system demand. The former rate decreases to much less than 2.5 mgd during periods of low demand; and the relatively narrow band of the rate of flow, as measured at the pumping station, is due to the stabilizing effect of storage tanks. The installation consists of two units, one of which can handle the load; the other is for standby duty.

One unit at the Park Street Pumping Station with a speed variation of 850–1,500 rpm has a capacity of approximately 1,800–3,000 gpm. Within this range, the efficiency of the pump is approximately 80–85 per cent. The combined overall efficiency of the pump and drive units varies between 62 per cent and 72 per cent.

* M-40, made by Foxboro Co., Foxboro, Mass.

Summary

Adjustable-speed centrifugal pumps are well suited for systems for which pipe friction is a predominant factor to be considered. Fewer adjustable-speed pumps than constant-speed pumps are required for a given job. The savings that result from the use of fewer units more than offset the initial higher cost of variable-speed drives. In addition, because of the improved operating efficiency of a modern adjustable-speed unit, substantial savings in power costs are possible.

Cleaning Methods for Deep Wells and Pumps

Claud R. Erickson

A paper presented on Sep. 22, 1960, at the Michigan Section Meeting, Traverse City, Mich., by Claud R. Erickson, Mech. Engr., Board of Water & Light, Lansing, Mich.

SMALL towns often find it more economical to obtain their water supply from wells, because their requirements are small, and expensive treatment and filtration can usually be dispensed with. Large cities that utilize wells generally are forced to do so by the lack of a suitable surface water source.

As the demand for additional water supplies steadily increases, it obviously behooves those communities utilizing wells to make the most efficient use of their existing facilities. This is just as important as the development of new wells in the effort to maintain adequate water supplies.

Two factors which are extremely important to the efficient operation of a well are: (1) stimulation—or, as it is sometimes called, "redevelopment"—of the well when it has declined in productivity, and (2) the cleaning and, if necessary, reconditioning of the pump.

There are four basic causes of declines in well production:

1. Lowering of the static water level
2. Inefficient pump operation caused by worn, corroded, or plugged parts
3. Deposits of scale, corrosion products, and microorganism growths in the well screen area and on the face of the formation
4. Fouling of the well screen area by mud, sand and silt.

Reconditioning of the well and pump can help in recovering lost production if the decline is due to any of the last three causes.

Deposits on Well Screens

Mineral deposits in wells and pumps, thereby reducing their capacity, are the result of a natural chain of events. Rain picks up carbon dioxide as it falls through the atmosphere, and the dissolved carbon dioxide is further increased by decaying organic matter in the soil as the rain water penetrates into the earth. The solvent power of this water is greatly increased by the addition of carbon dioxide, so that minerals such as bicarbonates, sulfates, and chlorides of calcium and magnesium are readily dissolved. Often present, although in smaller quantities, are compounds of iron, manganese, silica, nitrate, fluoride, and other elements.

Pumping a well reduces the static head in the vicinity of the well by an amount equal to the drawdown. This is caused by the resistance to flow of the water through the water-bearing formation. This reduction in pressure, plus an increase in water velocity, releases some of the carbon dioxide as a gas and thus reduces the ability of the water to carry the dissolved minerals in solution. Minerals such as iron hydroxide, calcium carbonate, and

magnesium carbonate are then deposited in the pump, in the well screen openings, in the gravel wall, and on the sides of a rock wall. Direct deposition of suspended matter from the water may also occur.

Careful design of the well and pump will help considerably to minimize deposition. Well screens of greater length and larger diameter will reduce entrance velocities and moderate pumping rates will hold the drawdown to a minimum. Mineral deposition on the upper portion of the screen will occur if the top is exposed by overpumping. To prevent this the pumping level, if possible, should be maintained at least 5 ft above the top of the screen.

Because there are several different types and designs of water wells and the composition of the deposits formed varies widely, it is apparent that no single treatment would be suitable for all wells. It is probably beyond the scope and capabilities of many municipalities to diagnose correctly the trouble in a well in which the production rate has fallen off, and then to prescribe the best and most economical treatment.

Definition of Stimulation

Stimulation is generally defined as the treatment of a well by mechanical, chemical, or other means for the purpose of reducing or removing an underground resistance to flow. The term as used in this paper does not include the actual reconstruction of a well by redrilling to a larger diameter or drilling to a greater depth. Neither does it include the treatment of a well for the purpose of clearing the produced water of sand, sediment, turbidity, or impurities introduced during drilling operations. Several methods of stimulation in common use are discussed below. An outline of their applications to various problems is presented in Table 1.

TABLE 1

Applications of Various Well Stimulation Methods

Method	Unconsolidated Aquifers	Consolidated Sandstone	Consolidated Limestone
Muriatic acid* followed by chlorine	removes iron, sulfur, and carbonate deposits	not usually effective	sometimes beneficial; best results obtained by pressure acidizing
Polyphosphate followed by chlorine	removes fine silt, clay, colloids, disseminated shale, and soft-iron deposits	not usually effective	not usually effective
Dynamiting	not recommended	effective for all types of well screen deposits	effective when very large charges are used
Compressed air	removes plugging deposits of silt and fine sand in areas adjacent to screens	not used	not used
Dry ice	same as compressed air	used, only rarely, to remove cuttings from the face of a new well	not usually effective
Surging	same as compressed air	rarely used	rarely used
Chlorine†	removes iron and slime-forming bacteria	removes iron and slime-forming bacteria	removes iron and slime-forming bacteria
Caustic soda	removes oil scum left by oil-lubricated pumps	removes oil scum left by oil-lubricated pumps	removes oil scum left by oil-lubricated pumps

* Not to be used with concrete screens.
† Usually used in a concentration of 500 ppm.

Surging

The process of surging involves the movement of water back and forth through the well formation in an effort to remove the fine particles of material clogging the well walls. There are several methods of surging, the simplest consisting merely of high-rate production alternating with complete pump stoppage. This is often accomplished by means of a timer which starts and stops the pump at frequent predetermined intervals. This is continued for at least 24 hr before a production test is run to determine whether or not any improvement has been made. Whether further surging is needed depends on the results of the test. A more violent and effective method is to use a wooden surge block piston approximately 1 in. less in diameter than the well. The piston is usually made of several layers of wood with layers of heavy canvas or sheet rubber in between. The latter are cut slightly larger than the wood and project so as to form a seal against the sides of the well. The block is secured to the well rig and moved up and down so as to create a vigorous movement of water through the well formation.

Other methods of obtaining surging action utilize compressed air, as in an air-lift pump, and dry ice.

Surging is often used in conjunction with chemicals to increase their effectiveness. Some of the chemicals used are phosphates, chlorine, acids, and wetting agents, the selection depending on the type of well and the nature of the clogging substances.

Vibratory Explosion

Vibratory explosion is a form of dynamiting in which the explosive is divided into many small charges and arranged to fire in rapid sequence, thus producing a vibrating effect. Gas released from the explosion creates a bubble which expands and contracts, gradually decreasing in size until it is finally damped out. The expansion and contraction are supposed to produce a strong surging action in the formation during the period when the vibrations are knocking particles and deposits loose. Some parts of the country, particularly Southern California, use the method extensively and apparently with very satisfactory results.

Acidizing Treatment

This is a popular and, in many cases, effective method of stimulating and cleaning wells. Muriatic acid, a commercial grade of hydrochloric acid which contains a small percentage of impurities, is normally used. The concentration usually injected into the well is 15 per cent.

An inhibiting agent should be added to the acid before use in order to delay attack on the metals in the well structure. Inhibitors form a microscopic covering over the metal and keep the acid away from it. In time (4–5 hr) the protection is broken down and the acid attacks the metal. For this reason it is important that the acid solution not be allowed to remain in the well for prolonged periods. Inhibitors slightly retard, but do not prevent, the reaction with iron or carbonate deposits.

Antifoam additives are now available to increase the effectiveness of the acid and prevent overflowing due to foaming.

Stabilizing chemicals are also frequently added to inhibited muriatic acid in order to prevent secondary precipitation of dissolved iron deposits. Without a stabilizing agent, the dissolved iron is likely to coagulate in a

gelatinous mass and settle to the bottom of the screen, thus resulting in plugging. Gelatinous ferric hydroxide will also form in the interstices of the sand and gravel outside the well screen and can sometimes result in almost total loss of production. Once such deposits have formed it is very difficult to remove them and restore the well.

Acid is especially well suited to the stimulation of drift wells where the screen is often subject to iron and carbonate deposits. Experience has shown that acid treatment is seldom beneficial for the stimulation of sandstone wells, although it may often be useful in the development of limestone wells. When used on limestone wells, it is advisable to treat the acid in order to delay action until penetration has been achieved.

Surging of the acid solution is normally accomplished by means of the pump. Where the static water level is low and there is sufficient volume in the column above the water level this can be done by closing the shutoff valve at the pump discharge and installing an air-release valve at the top of the column to expel air as the water rises. If there is insufficient volume in the column it is necessary to use an external tank.

Muriatic acid should never be used in a well equipped with a concrete screen or casing, as it will soften and disintegrate the concrete.

For treatment of wells of this type, citric acid is sometimes used. This is a much weaker acid than muriatic and a correspondingly large amount will be needed to remove deposits, resulting in a higher treatment cost. The approximate cost of these acids per hundred-weight is $7.50 for muriatic and $38 for acidic.

The acid concentration used for treatment is very important. A solution that is too weak will not be effective, whereas one that is too strong will displace or dissolve more material than can be carried in suspension or solution by the spent acid and result in heavy gels or precipitates with great plugging powers.

Pressure Acidizing

A recent development in acidizing borrowed from oil well techniques is pressure acidizing. It consists of injecting large quantities of acid solution into the well at high pressures, thereby producing deep radial penetration. The formation being treated must be sealed off from overlying formations or the acid will not penetrate radially far from the well. This method has not been used extensively, but it seems reasonable to assume that it might be quite effective in rock wells.

Polyphosphate Treatment

The use of polyphosphates for well stimulation is generally confined to drift wells. It can successfully be used for cleaning screened sand and gravel wall wells which are plugged with fine sand, silt, clays, colloids, and disseminated shales. The polyphosphates react with and hasten the removal of these materials, whether they are naturally found in the water-bearing formation or placed there during drilling operations. Polyphosphates can also be used to dissolve certain forms of soft iron deposits, and where these are present a considerable saving can be effected over the use of acid treatment.

Extreme care must be taken in the use of polyphosphates. Misapplications may result in great difficulties or disastrous failures. For example, they should not be used in the treat-

ment of wells with multiple screens in water-bearing strata separated by clay or shales. The action of polyphosphate on these materials will cause their disintegration and infiltration into the producing sands or gravel and further reduce the well productivity.

When detergents are used as a wetting agent in conjunction with polyphosphates better penetration of the formation sometimes results.

Polyphosphates range from very acidic to very basic. Proper selection should be made only after study of the well design, water analysis, well log, amount of production loss, and rate of loss.

Shooting Treatment

The use of dynamite (shooting) for stimulation is limited to rock wells and is accomplished by setting off charges of 50 per cent nitroglycerin dynamite at frequent intervals along the length of the water-bearing rock. A typical shot in a 14-in. sandstone well might use 5- to 10-lb charges every 5 ft. A recent innovation is the use of a string of high explosives which produces a linear explosion as distinct from a point explosion. Less material is removed in this way and the debris can therefore be cleaned out in a shorter time. Because plugging of the formation ordinarily occurs to a depth of only 1–2 in., this method should be just as effective as the point-blasting system, and experience seems to bear this out.

Larger charges are required in a limestone well, because limestone is less porous than sandstone and more surface area is needed. Blasting is usually required in the development of a new limestone well, the size of the individual charges ranging from 50 to 150 lb. Dynamiting should be done only by properly qualified persons. In some locations it is necessary to obtain police permits to handle and use dynamite. Local and state regulations should be checked.

In the Lansing, Mich., system of wells, seven methods of well stimulation have been tested, and dynamiting is now used exclusively. Tables 2 and 3 give data on the performance of dynamiting on Lansing wells. The advantages of dynamiting are clearly indicated by the saving in the cost of increased production over well reconstruction and extension.

Chlorination

Chlorine is sometimes used alone as a complete well treatment, but it is more frequently used as a supplement following an acid or polyphosphate treatment. It is almost always used as a final chemical treatment before putting a well back in service. A concentration of 50 ppm of available chlorine in the well provides effective disinfection, and a concentration of 200 ppm destroys certain types of iron bacteria. The destruction of sulfur and slime bacteria can be accomplished with chlorine concentrations of 500 ppm.

After the chlorine has been applied, the well should be surged with the pump several times every 30 min. At the end of 4 hr the well should be pumped to waste, or the chlorine solution will cause corrosion of the metal parts and oxidize the ferrous iron in the water to form insoluble ferric iron.

Chlorine and polyphosphate are sometimes used together in a common solution, and prepared mixtures can be obtained in this form. Chlorine, as purchased for use in wells, comes in a powder form as calcium hypochlorite.

TABLE 2

Performance of Successive Dynamiting in Lansing Wells

Item	First Dynamiting	Second Dynamiting	Third Dynamiting
Wells dynamited as of July 1959	55	38	6
Overall increase in specific capacity—*gpm/ft*	3.8	3.2	2.3
Avg drawdown after dynamiting—*ft*	66	67	73
Overall increase in yield—*gpm*	13,805	8,132	1,008
Avg increase in yield—*gpm*	251	214	168

Use of Compressed Air

Compressed air is occasionally used in drift wells to remove silt and fine sand from the area immediately surrounding the well screen. This is usually accomplished by means of double surge blocks located approximately 2 ft apart. The surge block assembly is lowered to the bottom of the screen and compressed air is admitted between the blocks. This confines the air and concentrates the blowing action. The blocks are subsequently raised in steps until the entire screen has been covered.

By using two pipes, one for water and one for air, an air-lift pumping action can be produced. The well can be surged in this manner, after which the proper valves can be operated and any sand, silt, or other loosened deposits can be pumped out.

Use of Dry Ice

Dry ice has been used to a rather limited extent in producing surging and also as a means of obtaining back pressure with a closed well head. Surging can be varied from light to heavy by changing the quantities of dry ice used. For light surging of a 6–10-in. well 10–15 lb would be used; 25–50 lb would be required for heavy surging. For larger-diameter wells proportionally larger quantities are re-

TABLE 3

Performance and Cost of Dynamiting

Well	Normal Drawdown *ft*	Increases Produced by Dynamiting		Cost of Dynamiting and Cleanout *$*
		Yield *gpm*	Specific Capacity *gpm/ft*	
1	171	256	1.5	1,454
2	48	96	2.0	1,586
3	21	160	7.6	1,122
4	52	130	2.5	1,653
5	60	234	3.9	1,387
6	78	179	2.3	1,204
7	55	160	2.9	1,779
8	58	157	2.7	1,773
9	64	339	5.3	1,176
All wells		1,711		13,135
Avg		190		1,459

quired. Care must be taken not to exceed 1 lb for every 11 lb of water in the well, or freezing will result.

Although the use of dry ice is an inexpensive and convenient method of agitating a well, it has been largely outmoded by more effective means of well stimulation.

Cleaning of Pumps

When the internal parts of a deep well pump become corroded or covered with deposits, loss of efficiency and pumping capacity results. The first indications of trouble are higher power consumption, indicating a loss in pump efficiency. This is invariably followed by reduced flows as the buildup of deposits continues.

If it is apparent from test data or daily log sheets that the pump requires attention and the unit has been in continuous service for a period of 3–4 years or more, it would probably be advisable to pull and dismantle the pump for cleaning and repair. If, however, the pump has been installed for a shorter period, and the nature of the deposit is known from past experience, it is often advantageous to clean the pump in place.

This is usually done with a 15 per cent concentration of inhibited muriatic acid (with a stabilizer added to hold the material in solution) or with a polyphosphate compound. Occasionally, both acid and polyphosphate treatments are used, particularly if the exact nature of the deposits is not known. The material is surged with the pump and discharged, to be followed by a 500-ppm chlorine solution.

It will be noted that the use of acid, polyphosphate, or chlorine for cleaning a pump in place is accomplished in the same manner as well stimulation with these chemicals. Thus the two operations are often performed simultaneously.

If a pump has been in operation for several years, or if there is reason to suspect excessive wear, the unit should be removed, disassembled, cleaned, and inspected. Worn parts should be repaired or replaced, and clearances adjusted.

If a large acid tank is available, the entire pump bowl assembly can be dipped in an acid bath and roughly cleaned before disassembly. Otherwise, the pump assembly should be scraped and wire brushed, after which it should be dismantled and the subassemblies cleaned. The parts which cannot be reached by brushing can be cleaned by dipping in a 15 per cent solution of inhibited muriatic acid. The column and shafting should also be thoroughly cleaned by brushing, scraping, and flushing.

It is often necessary to rebuild the bearing areas on the column shafting and pump shaft. This can sometimes be done by means of metal spray of the same type as the parent material.

After cleaning the column, the inside and outside should be protected with a bitumastic or other corrosion-resistant material which will not impart a taste to the water.

Well Analysis

The following procedure is given to illustrate the method which could be used in determining the proper treatment to increase the productivity of a failing well.

1. The static water level should be determined, as a drop in level could cause reduced production, higher pumpage costs, or even possibly cavitation, if the pump was drawing air. At least 1 hr should be allowed after shutdown before the level is measured.

2. The pump should be started and the pumping level taken. The drawdown thus obtained should be compared with the drawdown when the well was new. The pump should be operated at least 1 hr before measuring the pumping level. An increase indicates plugging of the formation or screen and the need for stimulation treatment.

3. With the pump discharge valve closed, the pump pressure should be taken and an attempt should be made to duplicate the manufacturer's pump curve. This is done by opening the discharge valve in steps and noting the pump discharge pressure and corresponding flow. Amperage or kilowatt readings should also be taken at these points to check on pump efficiency. For complete test information it may be necessary to disconnect the pump from the transmission main and install a temporary discharge line with a calibrated orifice for measuring flow. It is often possible, however, to obtain sufficient points with the pump connected to the transmission main if a suitable flow meter is installed. If the test data thus obtained indicate that the pump is operating at substantially less than rated efficiency, if the head-capacity curve is low, or if unusual noises are evident, the pump or column obviously requires attention.

A representative water sample should be drawn and sent to a laboratory for analysis. From this water analysis, and a knowledge of the type of well and aquifer, a recommendation can be made by experts as to the proper method of treatment.

Conclusions

In order to determine properly when a well should be stimulated, or a pump cleaned, it is necessary that adequate and continued data collection be made. It is especially important that full test data be obtained and permanently filed when the new well and pump are initially put in operation.

Well stimulation and pump cleaning can be economically justified whenever the cost of obtaining additional needed water supplies in this manner is less than the cost of securing an equal water supply by constructing new wells.

A large amount of data and experience has been accumulated by those specializing in water well stimulation. By a careful study and analysis it is possible to predict fairly accurately the method most likely to obtain the best results for a given well. Stimulation is not an exact science, however, and it is frequently necessary to arrive at the proper treatment by trial and error, particularly if full information regarding the nature of the aquifer is not available.

Acknowledgment

The author wishes to express his appreciation to the 100 water supply authorities and commercial firms who responded to the questionnaire on well cleaning, and to N. E. Gunderson, H. L. Keith, and Ervin Stahl of the Layne Northern Co., who contributed substantially to the article. Thanks are also due A. J. Lange, C. W. Dudley, and E. P. Darlington of the Lansing Board of Water and Light for their comments and aid in compiling the data.

Multiple Jacking of Concrete Pipe at National City

H. Arthur Price

A paper presented on May 19, 1960, at the Annual Conference, Bal Harbour, Fla., by H. Arthur Price, Sr. Project Design Engr., Dept. of Water & Power, Los Angeles, Calif.

AN unusual jacking operation was accomplished at National City, Calif., near San Diego, when 1,300 ft of 120-in. ID concrete storm drain pipe was jacked 1,300 ft during a multiple-jacking operation. The unusual features of the job consisted of a boring machine up front, four intermediate jacking stations, a rotary mud lubrication system, and a steering lead section of pipe. National City took bids for open-cut construction, with either precast pipe or monolithic construction. But an alternate bid made by a San Pedro (Calif.) contractor to do the job by jacking was accepted. The 75-ft cut, with only 100 ft of right of way, was suited for jacked casing because of the amount of shoring required and because the characteristics of the soil indicated that jacking would not be too difficult. The soil consisted mostly of clay, cobbles, cemented sand, and conglomerate.

Jacking Pit

Work began at a jacking pit (Fig. 1) that was constructed to hold four 250-ton jacks, each with a 13.5-ft stroke. Two-stage hydraulic pumps with 1,800 psi of pressure were required to do the job. The jacking pit had room for two sections of the drain pipe.

The jacking load was transferred to the pipe with a 0.75-in. steel ring faced with conveyor belt material to minimize damage to the pipe. Jacking operations at the pit advanced the pipe forward approximately 450 ft.

Boring and Excavation

At the front of the first pipe section, a rotary boring machine (Fig. 2) chewed away soil so that the jacks did not have to force the pipe through the ground. The machine was very firmly attached to the pipe section.

The boring machine had five arms. On each arm were a number of hard-faced teeth. The outer teeth could be turned to obtain a bore with a diameter larger than that of the pipe. The boring machine had a permissible forward travel of 20 in. When the machine advanced, the cutterhead was retracted and the pipe jacked ahead. As the machine rotated, the dirt was carried in veins, dumped into a conveyor belt, and then into a hopper 12 ft behind the machine.

All excavated material was hauled out of the heading by a battery-powered muck cart. The top of the cart was removable, so that the load could be hoisted out of the jacking pit by a derrick. The muck cart also served as a means of transporting ma-

terial, equipment, and drums of rotary mud to the various points of application.

Attached to the leading pipe was a steel cutting edge with a hard-faced tip. The outside diameter of the cutting edge was slightly larger than the outside diameter of the pipe, to provide for easier forward motion of the pipe. A manifold was incorporated with the leading section in which rotary mud was introduced at several points in the recess to lubricate the pipe as it moved forward. Also, 1-in. pipe connections for pumping rotary mud were installed at the joints of nearly every other section. An asbestos ring * was used to provide equal distribution of pressure over the ends of the concrete surfaces at each joint. The pressure was approximately 1,200 psi.

* Marinite, made by Johns-Manville, New York, N.Y.

Equipment and Installation

As was previously stated, pipe was jacked 450 ft from the jacking pit. To jack the pipe the remaining distance, four intermediate jacking stations were required. Three of the jacking stations were equipped with four 200-ton jacks; the fourth station had two 200-ton jacks. The intermediate jacks had a 48-in. stroke. The construction and installation of the jacking stations presented an unusual assembly problem.

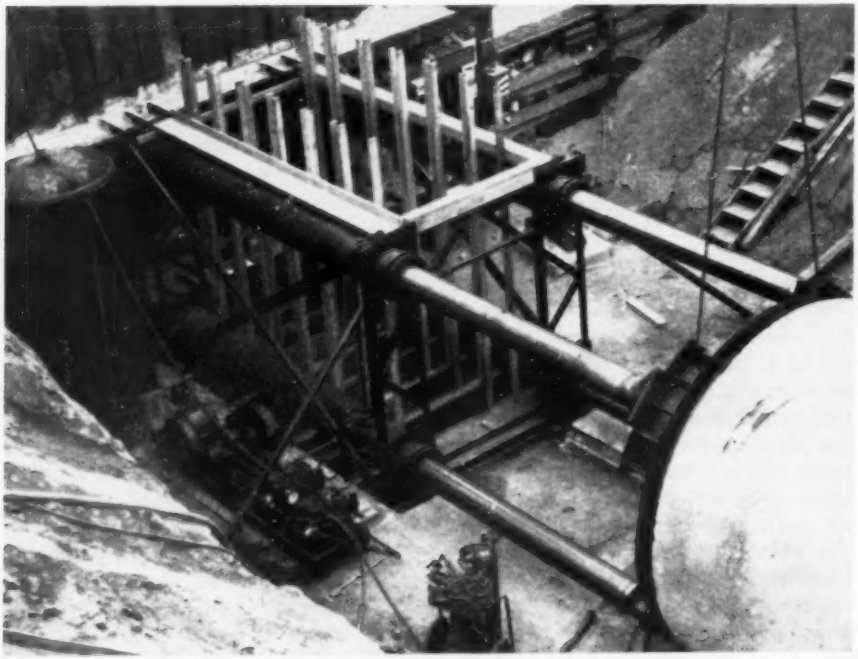

Fig. 1. Jacking Pit

Shown are the four 250-ton jacks against a section of pipe rimmed with a steel ring.

A section of pipe at an intermediate jacking station had a slightly recessed steel pipe sleeve that served to keep the soil from sloughing into the pipe. The pipe section contained a 10-in. channel separating the pipe joints. When the section was set in place, the channel was cut down the middle with a blowtorch, the steel frame for the jacks constructed, and the jacks subsequently installed. This work was done after the pipe section was installed.

The first four sections of pipe were tied together with four hog rods (Fig. 3) that were welded to the reinforcement in the pipe. These four sections directed the course of the pipe. At the trailing end of each hog rod were four 1-in. high-tensile bolts that could be tightened or loosened to affect the steering of the pipe. By relying on strain gages, operators soon learned if the pipe was getting out of alignment, and the necessary adjustments were made. On completion of the project, the pipe was found to be nearly on line.

Fig. 2. Rotary Boring Machine

The machine, firmly attached to the first section of pipe, has its cutterhead retracted as the pipe is jacked ahead.

Fig. 3. Hog Rods in Steering Section of Pipe

The four hog rods were welded to the reinforcement in the pipe. Beneath the two upper rods is the hopper behind the boring machine.

The jacks at the pit and intermediate stations were hydraulically operated. All other equipment, including the boring machine and the pumps for introducing rotary mud, was operated by air. All the jacking operations were accomplished by push-button control. The control station was at the front of the first pipe section, near the boring machine.

Lubrication

The project could not have been accomplished if rotary mud had not been used as a lubricant. The mud was formulated from bentonite, water, crude oil, and a jelled substance. It had the physical properties of minimum absorption of water and a uniform viscosity. The mud was introduced, by pumping, into the manifold at the cutting edge of the leading section of pipe. Rotary mud was also introduced at approximately every other pipe section. In addition, the

mud was introduced 50 ft in from the jacking pit by a gravity system.

Typical Operation

In a typical operation, the boring-machine operator notified the jacking operator when the boring machine had advanced to its limit of 20 in. The jacking operator pushed a button, and the leading string of pipe advanced and closed the distance. The jacking operator then pushed a second button, and the second string of pipe advanced and closed the distances between strings of pipe, and so on. Finally, he pushed a button and the last string of pipe, at the jacking pit, closed the distance to the previous string of pipe.

Work under the contract was performed 7 days a week, with three shifts per day. The pipe progressed approximately 5 ft per shift. The contractor, Johnson Western Co., San Pedro, Calif., conceived the means of jacking and designed most of the equipment. American Pipe & Construction Co. of California manufactured the 10-ft diameter concrete pipe.

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Underpinning a Section of Steel Pipeline

Harold L. Gunther and Dean C. Noll

A paper presented on May 19, 1960, at the Annual Conference, Bal Harbour, Fla., by Harold L. Gunther, Chief Engr., and Dean C. Noll, Asst. Engr., both of North Jersey Dist. Water Supply Com., Wanaque, N.J.

THE Ramapo force main is a steel pipeline built to increase the yield of the Wanaque Reservoir, at Wanaque, N.J., by 25 mgd. This is accomplished by pumping water through the main from the Ramapo River at Pompton Lakes, N.J., to the reservoir. The main is approximately 5 mi long. Construction was started in 1951 and completed in 1953.

The main can carry the full capacity (100 mgd) of the new Ramapo pumping station under a head of 115–135 ft. It was made in sections 40 ft long, except for curved and special sections, and was fabricated from two $\frac{7}{16}$-in. steel plates welded together longitudinally to form the 72-in. ID pipe. All transverse joints were butt welded in the field, except that a coupling * was substituted for the weld at approximately every 1,000 ft throughout the entire length of main to provide for contraction of the steel.

Course of Main

The force main starts at the discharge header in the Ramapo pumping station, is laid in a westerly direction in a borough street, crosses the Wanaque River in a concrete encasement, then runs north, paralleling the twin 74-in. Wanaque Aqueduct pipelines whose direction of flow is south. In its course northward through the Borough of Pompton Lakes and the Borough of Wanaque, the main crosses two rivers, passes under two railroads, through and across public streets, and, as it approaches the reservoir, passes through rugged hilly country. Excavation areas ranged from swamps and unstable earth to glacial till and hard granite rock.

The swamps gave the contractor the most trouble. The largest and only deep swamp is about 280 ft wide at the pipe crossing, with rock outcroppings at both sides and a steep rock cliff on the southerly end. Originally, several routes were surveyed to locate the pipe in a dry area, but because this swamp was so very long and wide it was decided to cross it at its narrowest point.

Installation Problems

In order for the contractor to bring in the pipe lengths, it was necessary for him to build a 3,700-ft access road through the rugged area. The road was built parallel and adjacent to the line of the pipe. The contractor dumped into the swamp approximately 4,000 cu yd of excavated rock and built the road on top of the rock. For several weeks, heavy trucks and equipment traveled over the road without causing any appreciable settling. After

* Made by Dresser Mfg. Div., Dresser Industries Inc., Bradford, Pa.

a conference between the contractor and the engineers, it was decided not to drive piling but to deposit more rock along the line on which the pipe was to be installed. After this was done and the area thoroughly compacted, cracked stone and a sand cushion were placed on top of the rock. The pipe was laid on the stone and sand and covered with 3 ft of embankment.

In March 1957, it was noticed that the access road and the pipe embankment had both settled. When levels were run along the top of the embankment, it was found that the pipe had settled considerably. Pumping was discontinued, the pipe emptied, and levels and line checked inside. The pipe was found to have settled 4 ft below the installed grade and to have shifted laterally 9 in. out of line at approximately the midpoint of the swamp.

About 80 ft south of the north edge of the swamp was located one of the couplings mentioned before. The pipe had pulled apart a distance of approximately 3 in. at the coupling without leaking. Joint harnesses were welded to the pipe across this joint to prevent further movement.

Support of Main

It was decided not to jack the pipe back to its original position but to underpin and secure it in its settled location. During the designing of the pipe-carrying supports, there arose the peculiar problem of attempting to bridge a swampy area 280 ft long with a 72-in. ID main that had a 3-ft earth cover. Previous experience showed that the supporting capacity of the swamp was poor; it was decided to discount this support entirely. The loads that had to be supported were

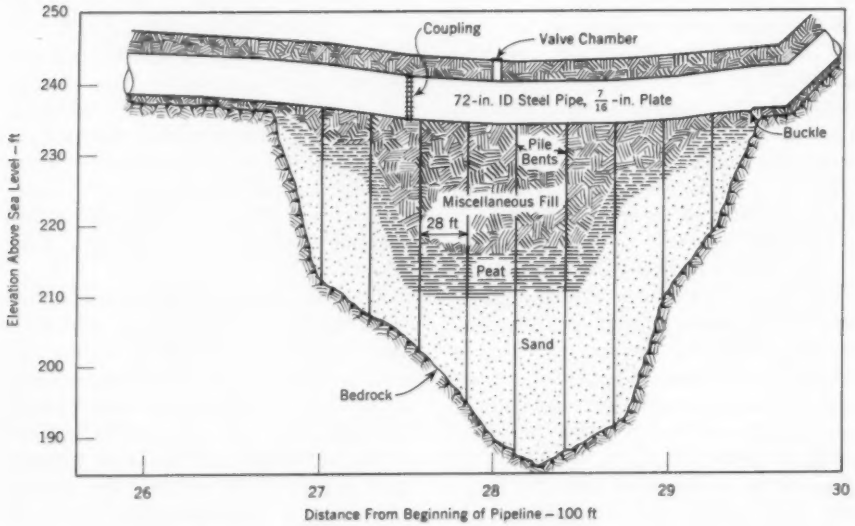

Fig. 1. Section of Main and Underlying Area

The pipe bridged a swampy area 280 ft long. Core borings 5 ft into the rock were taken to establish the rock's supporting capabilities.

the dead load of the pipe and the saturated earth fill, plus the live load of water and a dynamic load due to a vertical deflection in the pipe—a total of 4,200 lb/lin ft, or 1,176,000 lb for the entire 280-ft span.

To determine the nature and depth of the swamp, a contract was made with a drilling concern for a series of test borings adjacent to the pipeline. Eleven borings were taken in a space of 230 ft. The 2.5-in. casing was driven through the various strata of miscellaneous fill, peat, and fine sand and silt, to refusal on bedrock. Core borings were taken to a depth of 5 ft into the rock to establish the type of rock, its supporting capabilities, and the amount of its weathering. The borings showed that the greatest depth to rock would be 53 ft and that the piling would have a total unsupported length of 36 ft in the unstable material (Fig. 1).

Stress Analyses

When the nature and depth of the swamp were known, the beam strength of the pipe itself was examined to determine the spans that it was capable of withstanding. It was found that a pipe that could be kept round was a fairly efficient beam. But when computations were made for the spacing of the pipe saddles, the character of the problem radically changed.

The stress analysis of a pipe supported on saddles is difficult. In an attempt to compensate for unknowns or indeterminates, a small allowable unit stress is used. For an unstiffened pipe resting in saddle supports, high localized stresses occur at the saddle tips. According to Roark,[1] these stresses are less for a saddle subtending a large portion of the pipe circumference than for one that subtends a smaller portion of the circumference. For the ordinary range of dimensions, however, the stresses are independent of the width of the saddle—that is, the dimension parallel to the axis of the pipe.

In computations of the maximum span that the pipe could withstand, wide deviations were encountered when the formulas derived by various authorities were used. The maximum allowable spans for an unburied pipe ranged from a low value of less than 20 ft to a high value of 52 ft. With so much disparity among the authorities, it was decided to play safe and use the method that resulted in the smaller span.

As it was desirable to maintain the 3-ft earth embankment over the pipe, the total loading remained 4,200 lb/lin ft. A trial span of 10 ft was selected, and 120-deg saddles were used. The total stress at the saddle is the sum of the flexure stress in the span, one-fourth of the ring stress due to internal water pressure, and the localized stress at the saddle. The localized saddle stress, according to Roark,[1] is:

$$S = K \frac{P}{t^2} \log_e \frac{R}{t}$$

in which S is the localized stress at the saddle, in pounds per square inch; P, the total saddle reaction, in pounds; t, the pipe thickness, in inches; e, the base of natural logarithms; and R, the pipe radius, in inches. The factor K equals $0.02 - 0.00012\ (A - 90)$, in which A is the subtended angle of the pipe saddle, in degrees.

The 10-ft trial span and a loading of 4,200 lb/lin ft resulted in a total stress at the saddle of 17,400 psi. The span of 10 ft was too close for economical construction, inasmuch as the pile lengths could go to 50 ft. Other means were then considered, including a girder bridge on piles that would per-

mit close spacing of the saddle and still leave a reasonable span between pile bents. With this solution, however, the required girders for the bridge would be too heavy, because a 30-ft span was planned. Additional problems would have been encountered in attempting to support the pipe while the girder bridge was placed under it.

Uncovering the pipe to reduce the dead load was considered, but the drawbacks to this method overshadowed the desirable results that the point of support and, thus, the preservation of the pipe's beam strength. Schorer's analysis and presentation of ring girder design founded on the elastic theory were the basis of the ultimate design of this project.

The stress analyses included the investigation of stresses in the pipe wall and in the supporting ring girder. The direct longitudinal stresses in the pipe wall are the same as those in a hollow cylindrical beam, but the addition of the ring stress due to internal

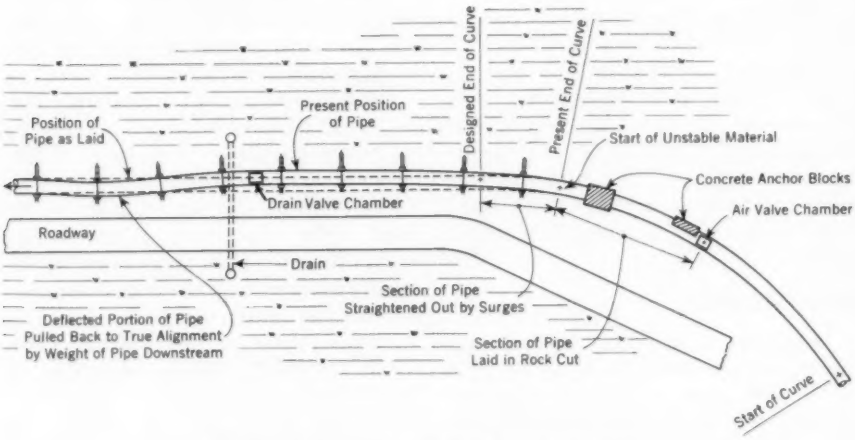
Designed End of Curve
Present End of Curve
Present Position of Pipe
Position of Pipe as Laid
Start of Unstable Material
Concrete Anchor Blocks
Drain Valve Chamber
Air Valve Chamber
Roadway
Drain
Section of Pipe Straightened Out by Surges
Deflected Portion of Pipe Pulled Back to True Alignment by Weight of Pipe Downstream
Section of Pipe Laid in Rock Cut
Start of Curve

Fig. 2. Movement and Settlement of Main

The lateral movement and settlement of the main were attributed to unbalanced loads.

could be obtained. The solution to the problem was found in Schorer's dissertation on ring girders.[2] Loads were computed once again, on the assumption that the pipe was buried under a 3-ft embankment with a maximum span of 30 ft between pile bents. This span was dictated by the load-carrying capacity of the particular H pile available.

The basis of the ring girder theory is the prevention of pipe distortion at pressure changes the principal stress system. An additional longitudinal stress due to rim bending at the point of support must also be considered. As the pipe in question was buried, temperature stresses did not play a large part in the analysis. Nevertheless, expansion was provided for in the form of slippage at the points of support.

The stresses in the supporting ring girder are the bending stress computed from the maximum moment by the

general bending formula, the ring stress due to shear forces, and the ring stress due to radial forces. These stresses are combined at the horizontal diameter, and the total maximum stress is the sum of the three.

In computations of the shearing stress due to radial forces, the area of the pipe wall under the ring girder was considered to act integrally with the ring girder. This area was thus added to the ring girder area to compute this particular shearing stress. Because it is believed that the maximum stresses occur when the pipe is half full, the maximum allowable stress for the pipe fully loaded was held to 10,000 psi.

Unbalanced Loads

Many theories were advanced as to why the pipe moved laterally and settled as it did. One theory held that the triangular shape of the fill, with the pipe on the outside edge and the road on the other side, coupled with the horizontal bend in the pipe, caused an unbalanced loading. This pushed the pipe further to the outside edge of the fill and increased the superimposed load on the fill. This, in turn, affected the angle of repose of the supporting fill, causing it to slide and allowing the pipe to settle (Fig. 2).

When the supporting pile system was being designed, consideration had to be given to the unbalanced horizontal load. This load consisted of a dynamic load due to a horizontal deflection in the pipe and a dead load due to the road's being higher and bearing against the pipe. This caused the pipe to act as a retaining wall. The unbalanced loads were compensated for by pile driving batter piles at a 30-deg angle with the vertical at every other pile bent (Fig. 3). Piles were first driven in at the prescribed locations, and sheeting was installed to permit excavation for the bents and ring girders. Because the swamp area was so unstable, the contractor was not permitted to excavate more than two holes at one time.

As a consequence of the variety of fill under and around the pipe, the pile driving did not proceed easily. The piles were thrown off line by logs and

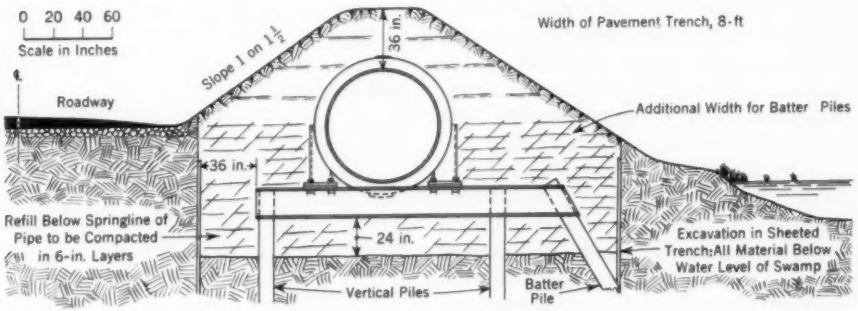
0 20 40 60
Scale in Inches
Slope 1 on 1½
36 in.
Width of Pavement Trench, 8-ft
Roadway
Additional Width for Batter Piles
36 in.
Refill Below Springline of Pipe to be Compacted in 6-in. Layers
24 in.
Excavation in Sheeted Trench; All Material Below Water Level of Swamp
Vertical Piles
Batter Pile

Fig. 3. Pile Bent Detail

Unbalanced loads were compensated for by the driving of batter piles at a 30-deg angle with the vertical piles at every other pile bent.

boulders, and considerable fitting was necessary to obtain the full bearing power of the piles.

Materials Used

Because the swamp was acidic, the piles were primed with coal-tar primer and then coated with coal-tar enamel. There was some concern about the ability of the pile coating to withstand the shock of the pile driver inasmuch as the pile driving was done in the cold of December and January. The results, however, proved the excellent bonding quality of the coating. There was no evidence of spalling, chipping, or shattering on the pile, except where the hammer made contact with the top of the pile. All of the steel used—including the ring girders, caps, struts, and plates—were coated in the same way as the piles.

It is recognized that greater spans could have been attained. But in this instance, the pile size and the unsupported pile length were the limiting factors. Heavier piles coupled with heavier ring girders could have permitted spans nearly to the point where the beam strength of the pipe would have been the limiting factor.

Construction was scheduled to begin in November 1957. The time factor was important, because the supplementary supply line was shut down, and the reservoir was fast approaching a record low. Large-size steel construction material was in short supply; the only pile size available in quantity was BP 10-57.

In the original design, wooden piles were provided for in all locations where it was believed they would be needed. Experience showed that it was extremely difficult to consolidate the deep swamp with fill; piles proved to be necessary. The settling of the pipe also proved the excellence of the butt-welded joints, which held tight, as did the coupling, even under the severe strain.

Acknowledgment

Illustrations were made from construction drawings by Robert Wieland, engineering assistant, North Jersey District Water Supply Commission, Wanaque, N.J.

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Metering and Meter Maintenance at Long Beach

Harold E. Wilson

A paper presented on Oct. 25, 1960, at the California Section Meeting, Long Beach, Calif., by Harold E. Wilson, Div. Engr., Water Dept., Long Beach, Calif.

TODAY it is almost universally accepted that the only truly equitable basis for charging for water service is through accurate measurement of the water used. As available supply of high quality, low-cost water approaches complete utilization, the trend to complete metering proceeds at an accelerated pace. This has certainly been true in Southern California, where the need for obtaining additional water is so well known that water conservation is recognized as a vital necessity for continued existence as well as future growth and development.

Unfortunately, many agencies have been slower to realize the necessity of setting up a program of routine meter maintenance and replacement. Such a program is essential if the true benefits of metering are to be realized. The accuracy demanded of water meters today necessitates very close tolerances, which, in turn, causes them to be affected by relatively little wear and very thin deposits. The practice of removing a meter only when the service pipe is enlarged or service is discontinued, or after it is apparent that the meter is registering inaccurately, can only result in a large loss of revenue.

This fact has been recognized by the Long Beach (Calif.) Water Department for a number of years, and, therefore, a real attempt has been made to set up a practical metering program. The AWWA Meter Manual,[1] recently published, very adequately covers the field of metering and meter maintenance. All persons charged with the responsibility of setting up or operating a metering program should familiarize themselves with the material presented in this manual.

Selection

The accurate measurement of the water served to any customer requires that the proper size and type of meter be installed. In Long Beach, the size of meter to be installed is determined by the building department, in accordance with the local plumbing regulations. The number of fixture units are totaled, and the proper size for the meter and house supply line is determined from tables prepared for various main supply pressures.

This usually results in a larger meter and house line than earlier installations. A multiple-unit dwelling of more than four units usually requires a 1½-in. meter by these standards. Larger apartments will often require a 2-in. service. Tests were run on six 1½-in. and five 2-in. meters, checking the accuracy at very small flows to see if an appreciable amount of water would be lost at flows consid-

erably less than the normal test rate. The accuracy obtained at rates as low as 0.25 gpm were very surprising, as shown in Table 1.

Installation

Much greater care and thought is given to meter installation when it is planned to replace the meter at regular intervals. One of the difficulties encountered in Long Beach is in servicing early installations where later replacements were apparently never considered. Much time is required to change a $\frac{5}{8}\times\frac{3}{4}$-in. meter which has been boxed in a 7-in. square concrete box. Unfortunately, Long Beach still has thousands of such boxes. They present a special problem in sidewalks or paved areas where they have been cemented in. These conditions are corrected during the routine meter exchange wherever possible, although replacement is not always practical without rebuilding the entire service.

Formerly, 1½- and 2-in. meters were boxed in 16-in. round steel boxes. Many are in concrete-paved areas and it is very difficult and costly to replace them with the larger concrete boxes

TABLE 1

Tests of New 1½- and 2-in. Nutating-Disc Meters

1½-in. Meter Flow Test Results

Meter No.	Test Rate—*gpm*					
	60	30	5	1.5	0.5	0.25*
	Actual Flow Registered—*per cent*					
3782377	100.1	100	100	100	100	99
3782378	100.1	101	100	100	100	99
3782379	100.1	101.2	100	100	100	99
3782380	100.2	101	100	100	100	99
3782381	100.5	101	100	100	100	99
3782382	100.5	101	100	100	100	99

2-in. Meter Flow Test Results

Meter No.	Test Rate—*gpm*					
	100	8	2	0.5	0.25	0.125*
	Actual Flow Registered—*per cent*					
3880451	100	100	100	100	98	94
3880452	100	100	100	99	97	40
3880453	100	102	100	100	97	80
3880454	100	102	100	100	99	89
3880455	100	100	100	99	98	89

* These flow rates were very accurately checked. The 0.25-gpm test of six 1½-in. meters took 31.5 min for a 1.0-cu/ft volume. The 0.125-gpm test of five 2-in. meters took 59.5 min.

now used, which are large enough to hold the shutoff valve. Difficult or time-consuming meter exchanges are often deferred to a more convenient time. If care is not taken, this more convenient time may never arrive. At present, a three-man crew is kept fully occupied going back over the system to service this type of installation. It is discouraging when two or three 1½-in. or 2-in. meter exchanges amount to a full day's work for a crew that costs more than $100 per day.

Compound meters, 3–10 in. in size, are installed in adequately-sized vaults with a flexible meter coupling on the outlet. This has been found to be more satisfactory than the grooved or bolted couplings formerly used. It has been possible to correct most of the large meter installations as meters are exchanged.

With accelerated meter exchange programs, it may be found desirable, for ⅝×¾- and 1-in. meter installations, to use one of the standard meter yokes available from several manufacturers. With increasing labor costs, the additional cost can be justified if the meters are exchanged at relatively short intervals.

Removal

The decision as to which meters should be removed was one difficulty Long Beach wrestled with for several years. Individual meter record cards were flagged with colored tape to indicate at a glance the year installed, but this, like other methods of marking individual meter cards, was found to be quite impractical. The most economical removal can be obtained only if meters are grouped together in the field. This was finally solved by setting up a separate address file where a single 5×8-in. card carried all the street addresses for one or more blocks, with the meter number and date of installation. A sample meter exchange record card is illustrated in Fig. 1.

The meter exchange crews take these cards with them in the field, checking the number of the meter found at each address and then filling in the make, number, and year of the exchange meter. The crew foreman also fills out a single-copy meter change order (Fig. 1). No carbon copies are made, as a single copy can be routed to all interested parties.

A three-man crew has been found to be most practical. To keep costs at a minimum, it has been found desirable to use good men and good equipment. It is common practice to assign older or poorer workers the routine tasks, but such men are often not productive and cause exchange costs to skyrocket. Last year, removing and resetting meters cost $34,172 for labor, $4,118 for material, and $3,561 for transportation, for a total of $41,851. This included individual nonregistering meters removed. The average costs per meter was $2.08 for ⅝×¾-in. meters, and $3.12 for 1-in. meters.

Reconditioning Program

A thorough meter reconditioning program has been set up at Long Beach. The exteriors of small bronze meters are sandblasted with outlets plugged with rubber stoppers to protect the measuring chambers. Meters are then disassembled and the main cases are returned to have their interiors sandblasted.

Measuring chambers are thoroughly cleaned to a bright, shiny finish by blasting with glass beads in a special blasting cabinet * which operates under

* Vacu-Blast Model 48; manufactured by Vacu-Blast Co., Belmont, Calif.

W-296 LONG BEACH WATER DEPARTMENT – **METER EXCHANGE RECORD**

MAIN LOCATION STREET [X] ALLEY [ ]

California Ave. ATLAS PAGE J-23

FROM 3501 TO 3599 STREET W/side

Address	Size	Make	Meter No.	Year	Size	Make	Meter No.	Year	Size	Make	Meter No.	Year
3501		T	4241060	54								
07		W	3845888	49	3/4"	W	4067008	58				
11		W	4059970	47	"	W	4166928	58				
15		T	4992233	54								
21		T	3396308	54								
31		W	4110949	60		T	1165497	57				
39	1"	T	4492235	54								
45	1"	T	3872803	54								
55		T	3949101	54								
59		T	2557765	53		T	366028	60				
69		W	4092806	48		W	4180620	58				
79	1"	T	4227258	54								
85	1"	T	3846207	54								
95	1"	T	2161009	47	1"	T	3704320	58				

W-306 Long Beach Water Department

METER CHANGE ORDER C

Service Address 3559 California Ave

Old Age [X] Noisy [ ] Non-Registering [ ] Meter Leak [ ] Special Test [ ] Change Dial [ ] Other [ ]

Remarks

REMOVED		INSTALLED	
Size	3/4"	Size	3/4"
Make	T	Make	T
Number	2557765	Number	366028
Reading	953	Reading	000

Communication Operators Order No. Date MAR 16 1960 Work Done By GALLOWAY

COMMERCIAL DIVISION

History Card	Meter Book	IBM

Billing Account No.

Fig. 1. Meter Exchange Record and Meter Change Order

Shown are a meter exchange record for the west side of California Ave. and a meter change order for one installation (circled on the exchange record). It should be noted that the record shows the change, in 1960, from a 10-year repair cycle to a 7–8-year period.

a partial vacuum (Fig. 2). This method of cleaning, which has been in use for more than a year, has resulted in an increase in production of approximately 47 per cent. More than 85 per cent of the glass beads are true spherical shapes; they are so fine they pass through a 140-mesh screen. The blasting cleans the chambers far more thoroughly and quickly than the inhibited muriatic acid formerly used (Fig. 3). Heavy or light corrosion products are easily removed, as the dry process affords fast cutting action and the cabinet window provides excellent visibility. The repairmen clean 16–24 chambers per hour.

The unit is charged with 8 lb of glass beads, and will clean more than 100 masuring chambers and discs before being replaced. At a bead cost of 25 cents per pound, cleaning costs less than 2 cents per chamber, which is less than half the cost of acid and inhibitor.

The introduction of this equipment has been the largest single step forward in the development of the meter maintenance program. All the meter repairmen continue to be enthusiastic over this method, which allows them to increase their production greatly and do a better job at lower cost. Previously, the average repairman repaired twelve $\frac{5}{8}\times\frac{3}{4}$-in. meters per day. The average for the past 12 months was 17.6 meters per man per day.

In evaluating the meter-reconditioning program, three factors should be considered: quality, quantity, and cost of repair work done.

Quantity of Repair

During the 1959–60 fiscal year, the Long Beach meter shop, with seven meter repairmen and a foreman, repaired 13,389 meters. Of these, 12,624 were $\frac{5}{8}\times\frac{3}{4}$-in. meters, repaired in 6,069.5 hr. A total of 1,153.5 hr was spent repairing and testing 1–8-in. meters. One man repairs all registers and train gears with an average daily production of 60 each.

Quality of Repairs

The quality of the repair work at Long Beach is best shown by the average flow test results for more than 1,000 $\frac{5}{8}\times\frac{3}{4}$-in. meters of each of three makes repaired during 1960 (Table 3).

The results compare very favorably with the results obtained with the last 1,000 new meters purchased: 12.0 gpm, 99.99 per cent; 2.0 gpm, 100.71 per cent; 0.25 gpm, 97.72 per cent.

All repaired meters, regardless of age, are required to test 90 per cent or higher at the 0.25-gpm flow rate.

Cost of Repair Parts

Care must be taken that parts are never replaced unnecessarily just in order to achieve high quality and quantity. There is no substitute for experience in deciding whether a corroded measuring chamber or a worn disc should be replaced. A measuring chamber for a $\frac{5}{8}\times\frac{3}{4}$-in. meter costs approximately \$8.25, and a disc piston \$3.29. Every mistake in judgment which results in an undue replacement of these two items costs \$11.54. If as many as 13,000 meters are repaired each year, it is readily apparent that replacement parts must be kept under close control. The first time Long Beach exchanged all $\frac{5}{8}\times\frac{3}{4}$-in. meters in the system, approximately 25 per cent of the chambers had to be replaced. During the last fiscal year, less than 5 per cent were discarded.

Last year, Long Beach spent \$33,061.83 for repair parts for 13,389 meters. Repairs on $\frac{5}{8}\times\frac{3}{4}$-in. meters cost an average of \$2.56 per meter, as shown in Table 3.

As a large number of three different makes were repaired, it is of interest to show the spread of costs for each (Table 3). Care must be used in interpreting these cost figures, as the exposure and history of each group are so different. Type A consists of meters 20–46 years old, in which a large number of chambers had been replaced in previous repairs. Type B meters are 7–21 years old and required more chambers, as fewer had been replaced during the previous exchange cycle. Type C meters are those removed from a portion of the system recently purchased that was served untreated well water which causes few corrosion problems.

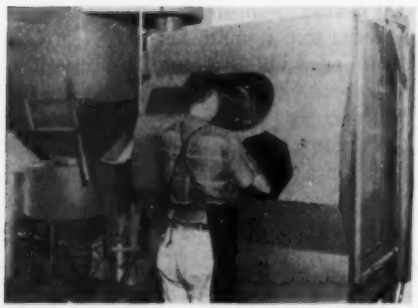

Fig. 2. Interior and Exterior of Blasting Cabinet

This method of meter cleaning has resulted in a 47 per cent increase in production.

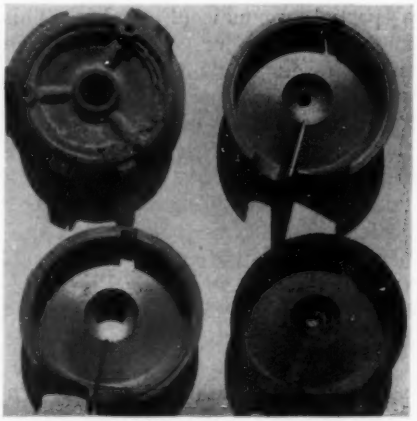

Fig. 3. Meter Measuring Chambers Before and After Blasting

With the blasting chamber, repairmen can clean 16–24 measuring chambers per hour.

Overall Repair Cost

Accurate records are kept to determine the true costs of meter repairs, including the costs of transportation, tool expense, depreciation, insurance, retirement, and employee medical plan. Table 4 presents the average cost for various meter sizes for the fiscal year ending Jun. 30, 1960.

The average hourly rates for last year, including allowance for holidays, vacation, sick leave, and retirement and insurance costs, were: meter shop foreman, $4.11; meter repairmen, $3.54; and shop men, $3.20–$3.72. Cost of parts includes 5–10 per cent handling charge.

Corrosion Control Before 1942

Corrosion of water meters presented no problem at Long Beach when the city was supplied entirely by local wells. During that period, a protective coating formed on the interior of the bronze measuring chambers and had no adverse effect on registration. The coating was easily removed by scrub-

bing with a bristle brush under flowing water. Measuring chambers after 20 years' exposure were still in very good condition, and few replacements were required. Cast-iron case compound meters did not present any serious corrosion problems; few tubercles formed, even on meters left in service for periods as long as 15–20 years. The interiors of cast-iron cases were cleaned by scrubbing and then replaced in the field with no further treatment.

Effects of Colorado River Water

In the Spring of 1942, Long Beach started to serve the western portion of the city with softened Colorado River water. This water was so different that shortly afterwards many meters were not registering. Sodium hexametaphosphate was added to the system at 3 ppm to delay or inhibit the afterprecipitation of calcium carbonate in the softened water, and to cause the gradual coating of outlying parts of the distribution system. This was discontinued as the system became stable.

After several years' exposure to softened Colorado River water, meters accumulated a hard blue-green deposit consisting of approximately 30 per cent copper sulfate, 60 per cent copper carbonate, and 2 per cent zinc carbonate. This deposit built up on the walls of the measuring chamber and in the ball socket until the meter either failed to register accurately or stopped completely. This hard deposit was difficult to remove except in an acid bath of inhibited muriatic acid. Irregularly shaped pits were found under these deposits, and, when the meters had been thoroughly cleaned and replaced in service, corrosion recurred at an accelerated rate at these points.

TABLE 2

*Average Flow Test Results for Three Types of $\frac{5}{8} \times \frac{3}{4}$-in. Meters, 1959–60**

Type of Meter	Test Rate		
	12.0 gpm	2.0 gpm	0.25 gpm
	Actual Flow Registered—*per cent*		
A	100.07	101.15	94.31
B	99.36	100.67	92.69
C	99.64	100.63	96.6

* More than 1,000 meters.

TABLE 3

Repair Costs for Three Types of Meters

Item	Type A	Type B	Type C
Average age—*years*	35	15	11
Time in service—*years*	7–8	7–8	6–15
Average repair cost—$	2.65	4.66	2.24

Effect of Well and River Waters

The Long Beach Water Treatment Plant was placed in operation Jun. 27, 1951. Treatment to remove taste, odor, and color, followed by filtration and postchlorination to obtain a residual of 0.85 ppm.

This treated water, blended with 5–20 per cent softened Colorado River water, now supplies all of the city, with the exception of the Harbor District and North Long Beach. Many difficulties were encountered similar to, but of lesser magnitude than, those experienced upon introduction of Colorado River water on the west side of town. This blended water was found to be much more corrosive than the well water, but not nearly so corrosive as the softened Colorado River water. Table 5 presents comparative analyses of the well water, blended water, and softened Colorado River water.

As these corrosion problems increased in magnitude, all galvanic couples were eliminated wherever possible. Small parts which cannot be made of standard cast bronze are made

of monel or phosphor bronze which is cathodic to, and therefore protected by, the large areas of cast bronze.

Early in 1955 it was evident that some form of protective coating should be applied to meter chambers. Because tin-plated chambers had been tried in the past with only limited success, it was thought an oil or grease coating offered the greatest promise. Laboratory tests were not considered desirable, as it is impossible to duplicate the corrosive and abrasive effects of water passing through meters in the field.

Three hundred pairs of meters were placed in the North Long Beach area, where the most severe corrosion had been experienced. Bronze measuring chambers were coated with several types of corrosion-resistant coatings. Each of the treated meters was installed with an identical untreated meter adjacent to it as a control. The average accuracy of the test meters was 99.8 per cent at 12 gpm, 101.8 per cent at 2.0 gpm, and 93 per cent at 0.25 gpm.

Treated meters with untreated control meters have been pulled at yearly intervals for tests and inspection. The results obtained to date are condensed in Table 6, which presents the average for all of the meters. Approximately three-fourths of the test meters have been examined, and the remainder will be checked annually. All the protective coatings have been found to be beneficial, and the protection afforded and accuracies obtained have greatly exceeded expectations.

TABLE 4

Long Beach Meter Repair Costs, 1959–60

Cost Item	12,624 ¾-in. Meters	503 1-in. Meters	27 1½-in. Meters	170 2-in. Meters	65 3–8-in. Meters
	Cost—$				
Supervision, depreciation, and miscellaneous costs	7,405.98	510.24	33.73	181.93	418.07
Shop work	4,883.10	336.42	22.24	119.95	275.65
Repairing	28,310.12	1,950.44	128.95	695.43	1,598.12
Testing	4,862.47	266.64	14.31	120.16	601.99
Registers and train gears	5,190.44	206.81	11.10	69.90	26.73
Total	50,652.11	3,270.55	210.33	1,187.37	2,920.56
Average cost per meter					
Labor	4.01	6.50	7.79	6.98	44.93
Parts	2.56	4.08	7.73	3.74	18.29
Total	6.57	10.58	15.52	10.72*	63.22

* Many of the 2-in. meters have been used as fire hydrant construction meters, which reduces the average cost of repair on this size meter because of the shorter time left in the field.

Current Corrosion Prevention

In 1955 the Long Beach Water Department had a total of 66,384 meters, representing a total investment of $1,143,473; therefore, it was desired to start an immediate program of corrosion control. A treatment using a low-viscosity, high-film strength oil *

* No-Ox-id C; a product of Dearborn Chemical Co., Chicago, Ill.

containing a series of inhibitors was selected. This was a product which had been used by several cities in California that have long had to deal with highly corrosive waters. This treatment has proved to be one of the best throughout the 5-year field test and is still used today.

After discs have been fitted to chambers and cleaned with glass beads, chambers are heated for approximately 15 min at 225°F to remove all moisture and then placed in metal baskets and immersed for 10 min in the oil bath heated to 160°F (Fig. 4). The oil is allowed to drain and the excess

TABLE 5

Average Analyses of Long Beach Water Supplies for Fiscal Year Ending Jun. 30, 1960

Constituent	Natural Well Water*	Treated Well Water and Softened Colorado River Water†	Softened Colorado River Water‡
	Concentration—*ppm*		
Silica (Si)	17	15	9.5
Iron (Fe)	trace	trace	trace
Calcium (Ca)	17.3	30.4	47
Magnesium (Mg)	1.5	4.7	15
Sodium (Na)	62	83	143
Potassium (K)	1	2	4
Carbonate (CO_3)	1	2	2
Bicarbonate (HCO_3)	172	166	132
Sulfate (SO_4)	12	77	263
Chloride (Cl)	20	44	78
Boron (B)	0.1	0.1	0.11
Fluoride (F)		0.4	0.4
Total dissolved solids	218	319	629
Hardness ($CaCO_3$)			
Total	50	95	179
Carbonate	50	95	112
Noncarbonate	0	0	67
Alkalinity			
Phenolphthalein	1	1	2
Methyl Orange	143	139	112
Free carbon dioxide (CO_2)	trace	trace	0
Color	39	6	4

* Specific resistivity 3011 ohms/cu cm; pH 8.3.
† Specific resistivity 2035 ohms/cu cm; pH 8.4.
‡ Specific resistivity 980 ohms/cu cm; pH 8.4.

At the start of this program, a group of meters was carefully tested before and after treatment, for it was thought the presence of the oil film might affect their accuracy at low flow rates. There was no evidence of any adverse effect on registration, however.

is wiped from the interior with an oil-saturated cloth.

The program was further expanded, early in 1956, to include all new meters even though it required disassembling, treating, and reassembling the meters before the acceptance test. Small disc

meters are now supplied by the manufacturer already treated to the department's specifications. A total of 56,000 meters was treated between August 1955 and October 1960.

is anodic to the bronze. For the past 8 years, the cast-iron bodies have been sandblasted to bright, shiny metal and covered with a plastic coating. The coatings have been found to perform

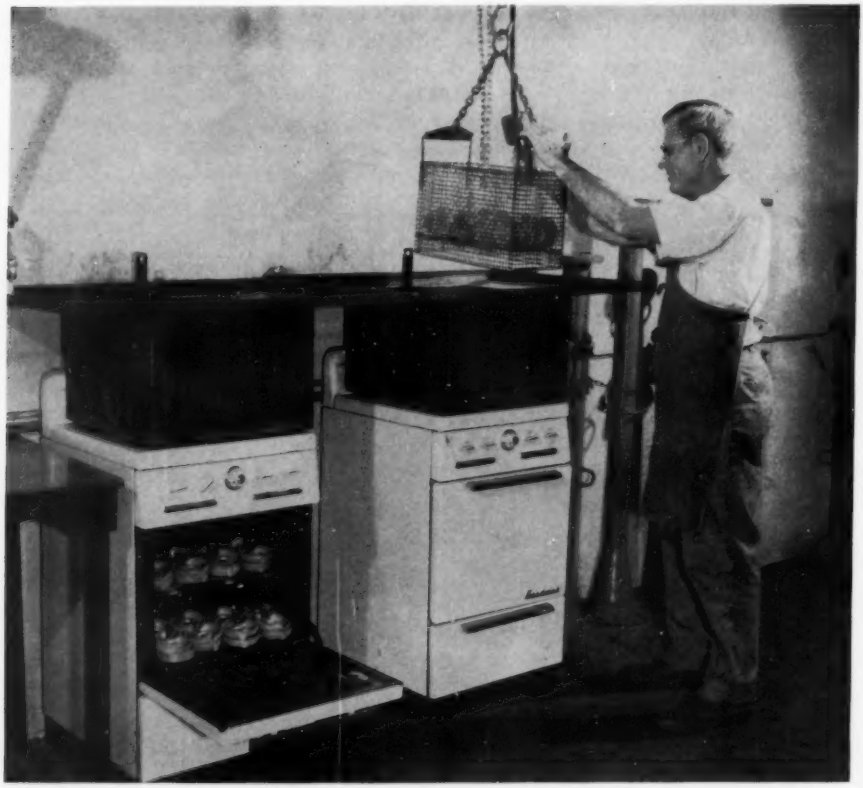

Fig. 4. Oil Treatment of Measuring Chambers

After having been cleaned by blasting with glass beads, measuring chambers are heated in the oven for 15 min at 225°F to remove moisture; they are then placed in metal baskets, as shown, and immersed in hot oil for 10 min.

Coatings for Cast-Iron Meters

Long Beach has a large number of compound meters with cast-iron cases and bronze interior fittings and measuring chambers. The cast-iron bodies are very severely attacked when exposed to corrosive waters, as the iron

very satisfactorily, only small tubercles forming at pinholes in the coating.

During this period, many new coatings have been developed which are vastly superior to the earlier coatings. Cast-iron interiors are now coated with a high–molecular weight epoxy resin

containing no solvents.* This material has extremely high dielectric strength and corrosion resistance. It has a very short pot life of approximately 15 min and must be applied by brushing it on the cast-iron cases which have been heated to approximately 125°F with an acetylene torch. A second coat is applied within 30 min to obtain molecular bond.

Where it is not practical to apply to smaller cast-iron bodies with inaccessible areas, a catalyst-cured epoxy resin containing solvents with a much longer pot life is used.† This coating is applied in three thin coats, alternating color, to insure complete coverage and elimination of all pinholes.

With either type of epoxy resin coating, the surface must be carefully prepared. All burrs, irregularities, and projections are ground to a relatively smooth surface, as epoxy resins are cohesive, rather than adhesive, before setting up. The surface is sandblasted to a clean, bright surface and the coating is applied the same day to prevent any rust from forming.

Protection of Bronze Case Meters

Compound meters with a main case made of the same bronze alloy used in the measuring chambers and interior fittings require much less protection. The meters are disassembled, the bronze interior parts removed, preheated, and then immersed in a heated corrosion-inhibiting oil. The bronze case is carefully heated to 150°–200°F with an acetylene torch on the exterior of the main case. The entire interior is then coated with the hot oil applied with a brush. The same procedure is followed with new meters before running the acceptance test. Careful testing before and after this treatment has revealed no significant change in registration.

TABLE 6

⅝ × ¾-in. Test Meters After 5 Years' Exposure to Softened Colorado River Water

Treatment, If Any*	No. Tested	Test Flow Rate—*gpm*				Condition of Chamber†
		12	2	0.5	0.25	
		Average Registration—*per cent*				
Corrosion-resistant grease with zinc oxide	47	101.9	103.0	98.6	91.9	E
Corrosion-inhibiting oil of high film strength	37	101.6	103.0	97.7	91.9	E
Microcrystalline wax and long-fiber petrolatum	35	101.4	102.4	97.1	91.6	E
Low-viscosity water-dispersing oil	40	101.1	102.9	97.2	89.3	E
Heavy-body water-dispersing oil	18	101.2	102.6	96.5	85.9	G
Friction-reducing oil	11	101.0	102.2	95.4	87.3	G
Clear lacquer	11	102.5	104.1	98.8	89.6	F
Average, all treatments		101.5	102.9	97.5	90.4	
Type A meter (untreated)	155	43.9	32.8	20.9	18.1	P
Type B meter (untreated)	69	45.7	41.9	23.1	16.3	P
Average for Types A and B		44.5	35.6	21.6	16.9	
Nickel bronze measuring chambers						
Untreated	37	95.6	92.5	82.8	65.2	F
Treated	24	101.9	104.4	100.1	81.5	E

* Standard bronze meters, except where noted.
† E, excellent; G, good; F, fair; P, poor.

* Specoat ETP; a product of Specialties Engineering Co., Pasadena, Calif.

† Soc-Co Plastic No. 7; a product of Soc-Co Plastic Coating Co., Paramount, Calif.

Test and Repair Frequency

Much has been written on the subject of determining the period a meter should be left in the field before removal for repair and test. Probably the most interesting presentation of this subject was by Richard V. Ford,[2] who gave this basic rule: "In any period of time, the highest net revenue from a metered water system is received when the cost of meter repair and maintenance just equals the loss of revenue if such work is not done." This is true, as repairing the meter too soon is as uneconomical as leaving an underregistering meter out too long.

The real problem is to determine the correct period of time. Unfortunately, there is no single answer, as the problem is very complex and depends on many variables, such as prevailing water rates, the corrosive effect of the water, the wear-out rate of the meters, the quality of repair work done, and the cost of removal, repair, testing, and installation. Further complicating the problem, the cost of repair varies with the length of time in the field. Obviously any arbitrary time interval will not be economically correct for all localities.

It would be of great assistance to many if the experiences at Long Beach could supply an answer, but this is a continuing study with many complications. Long Beach has large numbers of meters of three different makes, widely different ages, and exposures for varying periods of time to widely differing waters.

The benefits of corrosion control measures indicate that it is desirable to remove, repair, and treat all untreated meters as rapidly as facilities permit. This is especially true of the smaller disc meters, in which corrosion deposits have a more pronounced effect on registration. Many untreated meters removed have been found to be

TABLE 7

*Water Sales by Meter Size 1959–60**

Size of Service *in.*	Data Attributable to Service Size				
	No. of Services	Metered Consumption		Water Sales	
		Amount *1,000 cu ft*	Per Cent of Total	Amount *$*	Per Cent of Total
¾	69,691	1,260,532	60.3	3,019,996	67.0
1	6,990	212,823	10.2	502,022	11.1
1½	1,735	108,861	5.2	241,977	5.4
2	1,010	105,152	5.1	208,818	4.6
3	149	62,549	3.0	101,758	2.3
4	99	57,852	2.8	88,034	2.0
6	101	79,425	3.8	96,714	2.1
8	77	46,295	2.2	63,334	1.4
10	18	63,462	3.0	81,193	1.8
12	2	50,663	2.4	57,508	1.3
16	1	41,428	2.0	46,332	1.0
All sizes	79,873	2,089,042	100.0	4,507,686	100.0

* Water used by nonpaying city departments excluded.

accumulating extensive corrosion deposits even though the test indicates high accuracies at low flows. The life expectancy of these meters can be greatly increased if they are reconditioned and treated before becoming badly pitted. Measuring chambers of meters testing satisfactorily before disassembly have been found, on numerous occasions, to be so badly corroded that they had to be replaced.

At the present time, meters to be removed are selected by an alphabetical check of the meter exchange file for $\frac{5}{8}\times\frac{3}{4}$-in. meters set in or before 1953 and 1-, $1\frac{1}{2}$-, and 2-in. meters set before 1948.

Testing of meters removed from the field indicated the necessity of much more frequent reconditioning of the smaller meters. Many $1\frac{1}{2}$-in. and 2-in. meters were found to meet AWWA accuracy standards for new meters after 15–18 years in the field. The natural protection afforded by previous exposure to the old well water had made this possible. Newer meters set after introduction of the more corrosive softened Colorado River water, or the blended well and river water, have been more affected by corrosion.

All of the meters in the Long Beach system will have been treated with corrosion-inhibiting oil by the fall of 1962. As treated meters are brought in, the frequency of exchange will of necessity have to be reevaluated.

Evaluation of Current Program

During the past 9 years in which meters have been removed for periodic repair and testing, a total of 77,000 meters have been repaired. This program has accelerated from 5,312 meters in 1952, to a current rate of 14,245 meters. There has been no increase in meter shop personnel since 1952, although the average for the 3-year period 1952–54 was 5,561 meters.

In October 1960, Long Beach had a total of 83,081 meters, representing an investment of $1,889,191, with a replacement cost of $3,032,380. The protection of an investment of this magnitude justifies a program of routine maintenance and repair. The loss due to deterioration, however, and the resulting cost of repair or replacement parts are minor compared to the revenue lost from faulty registration or stoppage.

Table 7 shows the revenue realized through the sale of metered water in the city of Long Beach during the fiscal year 1959–60. It does not include the monthly service charge paid by each customer regardless of the amount of water used. The importance of maintaining small disc meters is evident when one realizes that 88.1 per cent of the water sales, for a total of $3,972,813, is through $\frac{3}{4}$–2-in. services. Each increase of 1 per cent in the average registration of all meters in the system would have increased revenue by $45,077 for the past fiscal year. An overall increase in registration of 3 per cent would have equalled the total cost of meter maintenance and repair, removal, and resetting in the field.

Table 8 shows the steady decrease in unaccounted-for water during the past 10 years. The record for the year 1959–60 of 3.93 per cent unaccounted-for water, with 95.75 per cent of the water to be accounted for metered, indicates that a very large gain has been made. The average for the first 3 years was 92.17 per cent metered and 6.92 per cent unaccounted for.

A further indication of the merits of an adequate metering program is the

TABLE 8

Long Beach Metered and Unaccounted-for Water

Fiscal Year	Total to Be Accounted for	Metered Use		Estimated Use*		Unaccounted-for Use	
		Amount *1,000 cu ft*	%	Amount *1,000 cu ft*	%	Amount *1,000 cu ft*	%
1950–51	1,512,834	1,393,687	92.12	6,512	0.43	112,634	7.45
1951–52	1,537,667	1,426,881	92.79	18,226	1.19	92,559	6.02
1952–53	1,639,585	1,502,057	91.61	18,235	1.11	119,292	7.28
1953–54	1,676,494	1,557,493	92.90	24,720	1.47	94,323	5.63
1954–55	1,717,465	1,602,245	93.29	26,816	1.56	88,403	5.15
1955–56	1,790,099	1,681,463	93.93	34,665	1.94	73,969	4.13
1956–57	1,883,289	1,746,328	92.73	57,014	3.03	79,946	4.24
1957–58	1,797,841	1,732,919	96.39	14,774	0.82	50,148	2.79
1958–59	2,075,600	1,948,675	93.89	8,315	0.40	118,519	5.71
1959–60	2,303,609	2,205,674	95.75	7,446	0.32	90,487	3.93

* Water used for fire fighting; flushing mains, streets, and sewers; chlorinating; and backfilling; and water lost by main and fire hydrant breaks.

large change indicated for the year 1958–59. The decrease in the percentage of metered water and the corresponding increase in unaccounted-for water reflect the faulty registration of approximately 13,000 meters acquired from a private water company which had virtually no meter maintenance program. All of these meters were replaced during the first 14 months. Judging from the number of high-bill complaints received following meter exchanges in this area, much revenue would seem to have been lost from faulty registration. The decrease in unaccounted-for water last year demonstrates the value of the exchange program.

Conclusions

The many factors which determine the optimum time to remove, recondition, and replace a meter vary with each water system. Changes in water quality, water rates, labor costs, and the efficiency and quality of meter repair procedures also affect the solution of the problem.

The use of a corrosion-inhibiting oil coating, which has virtually eliminated corrosion deterioration of measuring chambers and decreased wear as indicated to date, introduces another factor in arriving at the correct period for Long Beach. The most important single item to observe in evaluating a metering program is the metered percentage of total water to be accounted for.

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Determination of Nitrates by Ultraviolet Absorption

Eugene Goldman and Richard Jacobs

A contribution to the Journal by Eugene Goldman, Chemist, and Richard Jacobs, Student Engr., both of the Dept. of Water & Power, Los Angeles, Calif.

THE nitrate content of drinking water is of interest as an indication of pollution or bacterial activity and has become a matter of increasing significance since high concentrations of nitrate were linked to infant methemoglobinemia. In 1945, high concentrations of nitrates in well water in Minnesota were suspected as a cause of this disease.[1] The suspicions were realized when the 1947–48 epidemic in Minnesota resulted in 139 cases, 14 of which were fatal.[1] Although no case in the Minnesota epidemic was traced to a water containing less than 30 ppm nitrate nitrogen, 10 ppm was considered a significant concentration. Waring reported that "drinking waters containing high nitrate nitrogen—10–20 ppm or more—appear to be the cause of methemoglobinemia in infants. . . ."[2] In the light of these findings, an accurate means of determining nitrate is essential. The phenoldisulfonic acid (PDA) method will be reexamined here, and a new method for nitrate based on ultraviolet absorption will be presented.

PDA Method

The PDA method is the most widely used in the field of water analysis. Its shortcomings are well known, especially the extreme interference from the commonly present chloride ion. Chloride evidently reduces nitrate during treatment of the sample with phenoldisulfonic acid. The Robert A. Taft Sanitary Engineering Center at Cincinnati reported that, based on its Analytical Reference Service results, ". . . with correction for chloride interference the phenoldisulfonic acid method does not give reliable results for most operators."[3] A drinking water sample from Taft Center was distributed to 33 laboratories. The mean deviation for this sample to which 1.3 ppm NO_3^- was added was 27.8 per cent, when 27 of the laboratories used the PDA method.

An attempt was made to correlate percentage of nitrate reduction with amount of chloride present at two nitrate concentrations (Fig. 1). It has been found in the study of the PDA method that loss of nitrate because of chloride occurs in the nitration step, not during evaporation. As an exact procedure is not specified—the amount of base to be added or the evaporation time, for example—there may be random factors that also contribute to the lack of reliability of the PDA method.

Ultraviolet Absorption Method

The apparent course for the water analyst interested in more accurate nitrate results lies either in revising the PDA method or in trying something new. The reduction methods and the brucine method have been investigated by many analysts and found wanting. An extension of a method recommended by Hoather and Rackham [4] is offered as an alternative. This method depends on the absorption of nitrate in the ultraviolet region. Acidification to eliminate alkalinity interference and filtration to remove particulate matter are also included in the procedure.

1. General Discussion

1.1. *Principle.* The method is based on the absorption of ultraviolet light by nitrate ion in accordance with the Beer-Lambert law of absorption (Fig. 2).

1.2. *Interference.* Dissolved organic matter interferes with the method. The amount present can be estimated

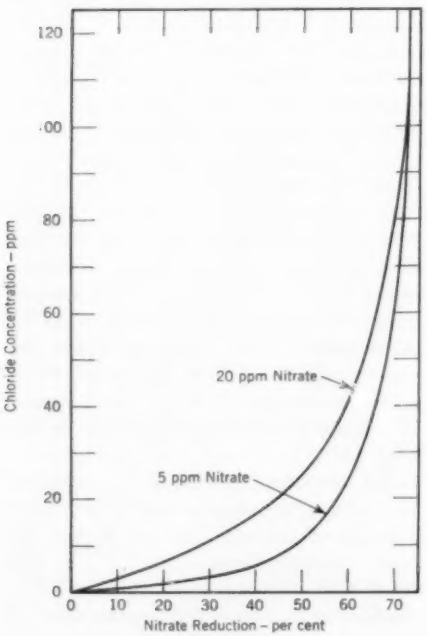

Fig. 1. Effect of Chloride on PDA Nitrate Determination

Shown is a correlation of percentage of nitrate reduction with amount of chloride present at two nitrate concentrations.

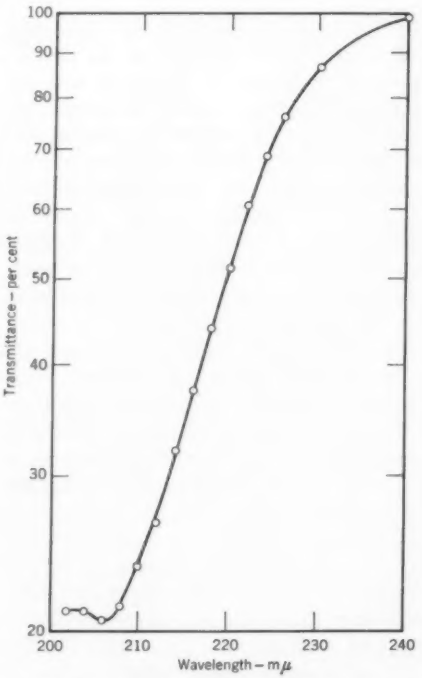

Fig. 2. Absorption Curve of 5 ppm Nitrate

The ultraviolet absorption method is based on the absorption of ultraviolet light by nitrate ion in accordance with the Beer-Lambert law of absorption.

by the total kjeldahl nitrogen determination. Nitrite and hexavalent chromium are severe interferences. Based on these findings, the method of correcting for organic interference which was suggested by Roather and Rackham [4] can be utilized (see Section 4).

High concentrations of hydroxide ion or carbonate ion interfere, but tially interfere. Negligible interferences were found from sulfate, ammonium bicarbonate, phosphate, and fluoride at the concentrations in which they are found in drinking water and most industrial waters.

As the light transmitted at 220 mμ is of high energy, error may arise from improper cleaning of the cells—especially from incomplete rinsing of di-

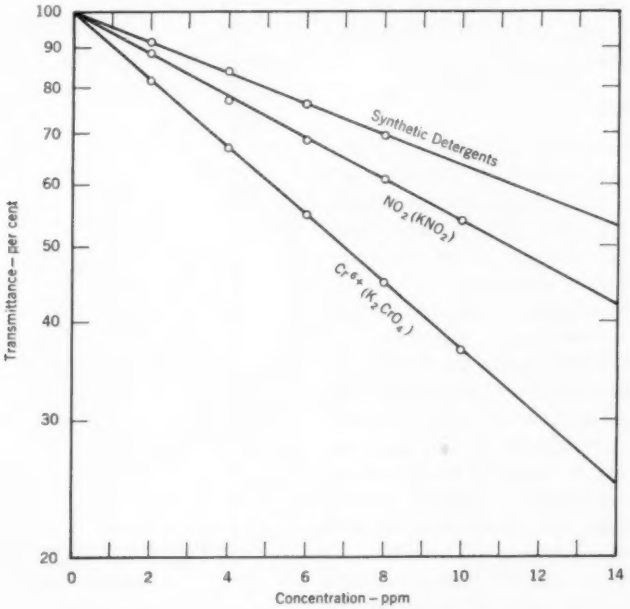

Fig. 3. Interference Curves: Absorption at 220 mμ

The synthetic detergents were 65 per cent alkyl benzene sulfonates.

acidification, as specified in Section 4.1, eliminates up to 1,000 ppm alkalinity (as $CaCO_3$). Chloride ion does not interfere.

The transmittance of sea water is 72.5 per cent at 220 mμ, equivalent to 2.5 ppm nitrate. It can thus be concluded that a multitude of common ions in high concentration do not substantluted cleaning solution or from insufficient cleaning of streaks or particles from the outside of the cuvets. Error can also come from incomplete rinsing of synthetic detergents from nessler tubes or cuvets. As an added precaution, possible interferences of particles larger than 0.45 μ are eliminated by filtration.

2. Apparatus

2.1. *Spectrophotometer,* for use at 220 mμ and 275 mμ.*

2.2. *Filter.* Type HA filter † or equivalent is used with appropriate filter assembly.

2.3. *Nessler tubes,* 50-ml, short-form.

3. Reagent

HCl solution, 1N. To 20 ml of 6*N* nitrate-free HCl, add 100 ml of redistilled water.

4. Procedure

4.1. *Preparation of sample.* Filter 50 ml of sample through the filter assembly into a 50-ml short-form nessler tube. Add 1 ml of 1*N* HCl, and shake the acidified mixture to insure complete reaction of carbonate.

4.2. *Transmittance and absorption.* Read the percentage transmittance, against redistilled water set at 100 per cent transmittance, of the sample at 220 mμ and then at 275 mμ. As nitrate does not absorb at 275 mμ, absorption here is assumed to be due to organic interference. Organic matter absorbs about 2.5 times more at 220 mμ than at 275 mμ. Therefore, to convert organic matter into equivalent nitrate, read the nitrate value of the organic absorption at 275 mμ from the standard nitrate curve drawn at 220 mμ. This value is multiplied by 2.5 and subtracted from the gross nitrate value read from the 220-mμ curve. Where sample organic load is known to be negligible, this correction may be omitted. Correction must be made for high concentrations of nitrite and for chromium or synthetic detergent concentrations (Fig. 3).

4.3. *Standard curve.* To obtain the standard curve, run standards at 220 mμ using 50-ml aliquots of redistilled water containing 0.00 mg, 0.05 mg, 0.10 mg, 0.20 mg, 0.30 mg, . . . 1.50 mg nitrate (as NO_3^-). Add to each standard 1 ml of 1*N* HCl, mix, and read percentage of transmittance at 220 mμ. Using 2-cycle semilog paper, plot the percentage of transmittance against

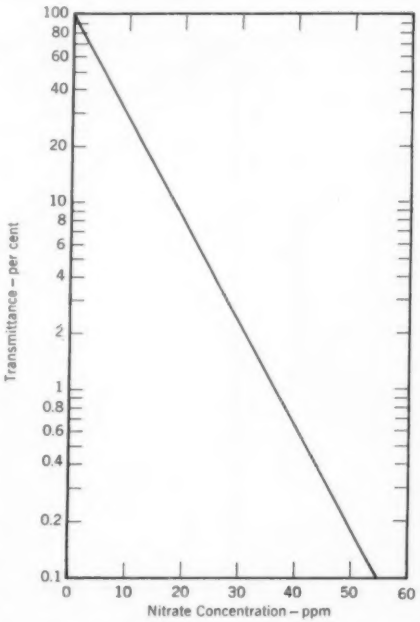

Fig. 4. Standard Nitrate Curve at 220 mμ

The percentage of transmittance was plotted against nitrate concentration on 2-cycle semilog paper.

* The data were obtained with a Beckman Model DU spectrophotometer, with a photomultiplier attachment and hydrogen lamp source. The slit size used was 0.3 μ.

† Millipore filter, made by Millipore Filter Corp., Watertown, Mass.

nitrate concentration in parts per million (Fig. 4).

5. Calculations

5.1. Concentration of nitrate in parts per million can be read from the standard curve (see Procedure).

5.2. If ionic interference is present, the effective concentration of interference may be calculated from the interference curves.

Acknowledgment

The authors wish to thank William R. Ree Jr., of the Los Angeles Department of Water and Power, for preparing the illustrations.

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Analysis of Distribution Networks by Balancing Equivalent Pipe Lengths

Annabel L. Tong, T. Francis O'Connor, Donald E. Stearns, and William O. Lynch

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A METHOD is presented here of sizing mains in water distribution networks directly, rather than by trial-and-error selection of pipe sizes that is inherent in conventional methods of analysis. The method was originally suggested by O'Connor in 1944, after publication of his simplified method of analyzing water distribution systems.[1]

Although it includes an iterative procedure similar to that in the Hardy Cross method,[2] the new method successively adjusts assumed flows in the network to balance relative pipe resistances rather than head losses. (As will be demonstrated later, pipe resistances may be expressed as equivalent lengths of a single size of pipe of uniform internal roughness.) Furthermore, an advantage of the method is that it may be used in the design of a distribution network to select directly the most economical sizes of mains to fit the head loss and flow requirements established by topography, consumption, and fire protection needs.

Two illustrative examples will be presented to demonstrate the application of the method and the conditions under which which it may be advantageously used. Two nomographs are included to facilitate calculations.

Method of Analysis

The Hazen-Williams formula states the relationship between water flow and head loss in a pipe as:

$$H = \frac{LQ^{1.85}}{17{,}076C^{1.85}D^{4.86}} \quad \ldots\ldots\ldots (1)$$

in which H is the head loss, in feet; L, length of pipe, in feet; Q, rate of flow, in gallons per minute; C, a coefficient indicative of the internal smoothness of pipe; and D, diameter of pipe, in feet. To simplify Eq 1, a new term, L_e, is introduced:

$$L_e = L\left(\frac{100}{C}\right)^{1.85}\left(\frac{0.667}{D}\right)^{4.86} \quad \ldots (2)$$

in which L_e is the length of 8-in. (0.667-ft) diameter pipe with C having a value of 100, which is the hydraulic equivalent of a pipe whose characteristics are L, C, and D. Hence, Eq 1 becomes:

$$H = \frac{L_eQ^{1.85}}{119 \times 10^5} \quad \ldots\ldots\ldots (3)$$

Equation 3 defines one of the hydraulic conditions that must always

prevail in each pipe in a fluid network. A second condition is that in a closed circuit or loop of pipe, the algebraic sum of the head losses in the individual sections of pipe, as determined from Eq 3, must always total zero:

$$\Sigma H = 0 \quad \ldots\ldots\ldots\ldots\ldots (4)$$

For the simultaneous solution of Eq 3 and Eq 4, the conventional Hardy Cross method calculates and successively applies correction factors to the assumed flow quantities in each loop until a satisfactory balance has been achieved. In order to do so, fixed values of L_e are substituted in Eq 3. If the head losses in the network prove to be excessive, new values of L_e must be tried, and the whole procedure repeated.

In view of the practical requirement that a balanced network utilize only the available head, the method presented here fixes values of H in Eq 3 so as to satisfy Eq 4 directly, according to the maximum head available from point to point in the system. Furthermore, maximum utilization of all of the available head normally will result in a design that requires the least amount of pipe. For a network of pipe of a single size, the "least amount of pipe" corresponds to the minimum possible total of the equivalent lengths of all the pipe in the network. This condition has been observed to occur when the algebraic sum of the equivalent lengths of the pipe in each and every loop in the network equals zero:

$$\Sigma L_e = 0 \quad \ldots\ldots\ldots\ldots\ldots (5)$$

Equations 3 and 5 may be solved by iteration until a balance is achieved. After this balance, the L_e values thereby determined may be substituted in Eq 2, together with C values and actual lengths (L), to determine the required diameters of the several sections of pipe in the system.

Imposing Eq 5 as a required condition in the design of a water distribution network also tends to establish a general evenness of flow throughout the system. In application, the evenness is distorted somewhat by the frequently found non-uniformity of occurrence of normal daily consumption throughout the network and by the practical necessity of representing consumption, in the mathematical analysis of the system, by a number of concentrated drafts located at selected pipe junctions. Nevertheless, a general evenness of flow in a system, which means that all available routes are used to maximum effectiveness in carrying both normal demand and fire demand, is a desirable characteristic of good distribution system design. If evenness of flow prevails under the most critical conditions of fire demand, the system generally will be adequate to meet fire demands in less critical locations or in less critical quantities, or both. Accordingly, the method outlined in the previous paragraphs is viewed as producing the optimum design for any set of fixed conditions of topography, pressure requirements, source of supply, draft, and geometric pattern of distribution network.

To calculate the equivalent lengths of pipe by this method, initial flow rates in each pipe in a system are first assumed. The head loss in each pipe is determined by the allowable pressure head and topographic elevation at various locations in the system. The assumed flows are then changed by the amount of their calculated correction factors, and the process is repeated until the equivalent lengths of pipe are balanced in each and every loop in the system.

If the first-assumed flows in the individual sections of pipe in a loop are designated as Q_1, Q_2, . . . Q_n, then, from Eq 3, it may be seen that the sum of the equivalent lengths of n lines in a loop is:

$$\Sigma L_e = f(Q_1, Q_2, \cdots Q_n) = 119 \times 10^5 (H_1 Q_1^{-1.85} + H_2 Q_2^{-1.85} + \cdots H_n Q_n^{-1.85}) \ldots (6)$$

To satisfy Eq 5, a correction factor ΔQ must be added algebraically to the initially assumed flows in the loop. Hence, Eq 6 becomes:

$$f(Q_1 + \Delta Q, Q_2 + \Delta Q, \cdots Q_n + \Delta Q) = 0 \ldots (7)$$

By the use of the first two terms of Taylor's series, Eq 7 may be expanded to:

$$f(Q_1, Q_2, \cdots Q_n) + \left(\frac{\partial f}{\partial Q_1} + \frac{\partial f}{\partial Q_2} + \cdots \frac{\partial f}{\partial Q_n} \right) \Delta Q = 0 \ldots (8)$$

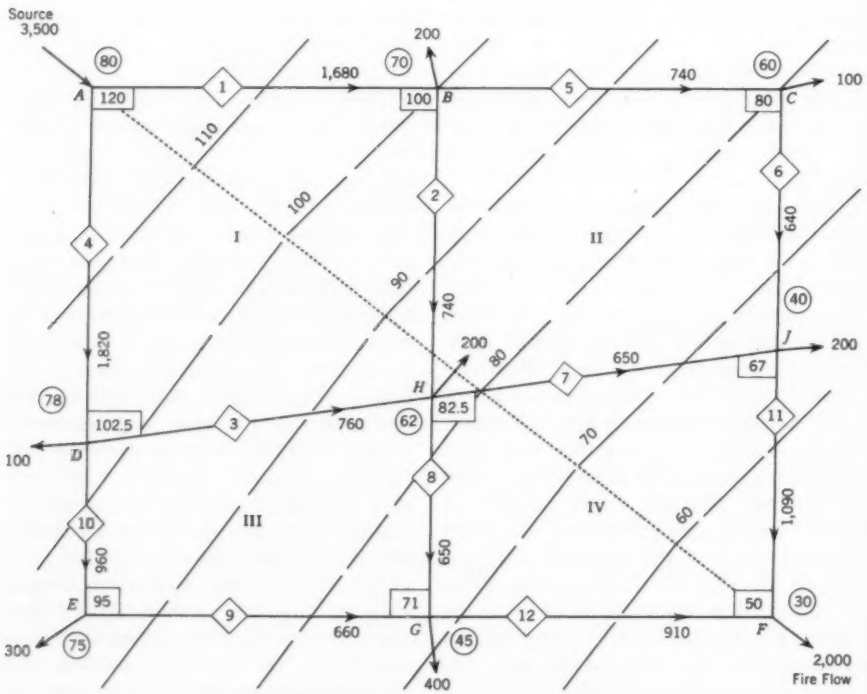

Fig. 1. Street Plan of System in Example 1

Boxed numerals represent hydraulic elevations (in feet), circled numerals, topographic elevations (in feet); and numerals adjacent to arrows, flows (in gallons per minute). Letters refer to points in the system discussed in text. The dotted line from A to F represents the maximum slope of the hydraulic gradient between the two points. The dashed lines are hydraulic contour lines showing head, in feet. Loops are designated by roman numerals; pipelines in each loop, by diamond-enclosed numerals.

From Eq 6, it may be shown that:

$$\frac{\partial f}{\partial Q_i} = -1.85 H_i Q_i^{-2.85} = -1.85 \left(\frac{L_e}{Q}\right)_i \quad \ldots (9)$$

When Eq 9 is substituted for parts of Eq 8, the correction factor is solved as:

$$\Delta Q = \frac{\Sigma L_e}{\Sigma 1.85 \left(\frac{L_e}{Q}\right)} \quad \ldots (10)$$

The sign convention for Q is established as plus (+) for clockwise direction and minus (−) for counterclockwise direction of flow in a loop. H and L_e have the same signs as Q in any pipe. ΣL_e is the algebraic sum of the L_e values for all sections of pipe in a loop. $\Sigma \left(\frac{L_e}{Q}\right)$ is the arithmetic sum of the quotients, $\frac{L_e}{Q}$, for all sections of pipe in a loop. ΔQ, calculated as $\frac{\Sigma L_e}{\Sigma 1.85 \left(\frac{L_e}{Q}\right)}$ for each individual loop, should be added algebraically to the assumed Q in each pipe in the loop in question. Each interior pipe, however, will also have applied to it a correction factor derived from the loop adjacent to the loop in question, but the ΔQ from this adjacent loop must be subtracted algebraically from the Q in the common pipe.

Mathematically, Eq 10 does not give an exact correction for Q, but successive applications of this equation will converge rapidly toward an acceptable solution—that is, one in which ΣL_e approaches zero in each loop.

Application

Example 1. The computation procedure for the method when applied to a simple pipe network of four loops was designated Example 1. To design the pipelines and to distribute the flows in this system, these steps were taken:

A street plan of the system was prepared with water main locations and topographic elevations (circled numerals) marked thereon (Fig. 1). The hydraulic elevations at the location of the source of supply (Point A) and at the location of maximum fire demand (Point F) were designated on the plan. The hydraulic elevation at A was given as 120 ft. When a desired minimum residual pressure of 20 ft of head at all intersections was assumed, the hydraulic elevation at F was calculated to be 50 ft, which equals its true elevation of 30 ft plus the required residual pressure of 20 ft of head.

The hydraulic elevations at other intersections in the system were determined in the following manner: The maximum slope of the hydraulic gradient from A to F, shown as a dotted line in Fig. 1, was assumed to follow a straight line between the two points. Hydraulic contour lines at approximately equal intervals were delineated on the entire grid. The contours preferably should be made parallel straight lines, but when the topographic elevations are irregular, care must be taken in locating the contours to preserve the minimum desired residual pressure. On hilly sites, it may be impossible to obtain hydraulic gradients having evenly spaced parallel contours.

For the network in Fig. 1, the shortest gradient, AF, was divided into seven equal sections, each section representing 10 ft of head loss along this route. Ideal contours, which would usually result in the most economical solution, would be parallel lines perpendicular to Line AF. As the required hydraulic elevations at the dis-

TABLE 1

Computation Sheet for Example 1

1	2	3	4	5	6	7	8 (1st Correction)				9 (2nd Correction)				10	11	12
Loop	Pipe-line	H *ft*	Q *gpm*	L_e *ft*	$1.85\frac{L_e}{Q}$	ΔQ *gpm*	Q *gpm*	L_e *ft*	$1.85\frac{L_e}{Q}$	ΔQ *gpm*	Q *gpm*	L_e *ft*	$1.85\frac{L_e}{Q}$	ΔQ *gpm*	L *ft*	C	D *in.*
I	1	20.0	1,720	250	0.3	−20	1,700	260	0.3	−20	1,680	260	0.3		2,000	100	12.2
	2*	17.5	770	960	2.3	−20+0	750	1,000	2.5	−20+10	740	1,050	2.5		1,750	100	8.9
	3*	−20.0	−750	−1,150	2.9	−20+20	−750	−1,150	2.9	−20+10	−760	−1,100	2.7		2,015	100	9.1
	4	−17.5	−1,780	−210	0.2	−20	−1,800	−200	0.2	−20	−1,820	−200	0.2		2,000	100	12.8
		0		−150	5.7	= −27		−90	5.9	= −15		10	5.7	=2			
II	2†	−17.5		−960	2.3	0		−1,000	2.5	−10		−1,050	2.6				
	5	20.0	750	1,150	2.9	0	750	1,150	2.9	−10	740	1,200	3.0		2,000	100	8.9
	6	13.0	650	970	2.8	0	650	970	2.8	−10	640	1,000	2.9		1,500	100	8.6
	7*	−15.5	−670	−1,100	3.0	0+20	−650	−1,180	3.4	−10+10	−650	−1,180	3.4		2,015	100	8.9
		0		60	11.0	=5		−60	11.6	= −5		−30	11.9	= −3			
III	3†	20.0		1,150	2.9	−20		1,150	2.9	−10		1,100	2.7				
	8*	11.5	650	860	2.5	−20+20	650	860	2.5	−10+10	650	860	2.5		1,250	100	8.6
	9	−24.0	−630	−1,880	5.5	−20	−650	−1,800	5.0	−10	−660	−1,750	5.0		2,000	100	8.2
	10	−7.5	−930	−290	0.6	−20	−950	−280	0.6	−10	−960	−270	0.5		1,000	100	10.5
		0		−160	11.5	= −14		−70	11.0	= −6		−60	10.7	= −6			
IV	7†	15.5		1,100	3.0	−20		1,180	3.4	−10		1,180	3.4				
	8†	−11.5		−860	2.5	−20		−860	2.5	−10		−860	2.5				
	11	17.0	1,120	460	0.8	−20	1,100	480	0.8	−10	1,090	480	0.8		1,500	100	10.1
	12	−21.0	−880	−840	1.9	−20	−900	−860	1.8	−10	−910	−840	1.7		2,000	100	9.5
		0		−140	8.2	= −17		−60	8.5	= −7		−40	8.4	= −5			

* Interior pipeline common to two loops, in the first loop in which it appears.
† Interior pipeline common to two loops, in the second loop in which it appears.

tant corners C and E were found to be 80 ft and 95 ft, respectively, the hydraulic contour lines for 80 ft and 90 ft were bent slightly to satisfy the minimum pressure requirement of 20 ft. The remaining contour lines may be fanned out to keep the head losses from the source to any intersection along any route approximately proportional to the distance from the source to the intersection. It is possible to work out more than one set of contour lines to satisfy the pressure requirements in a given system. One set of contours may offer better pressures in certain areas; other contours will favor other areas. The final solution will depend on the engineer's judgment.

Hydraulic elevations at intersections, based on the assumed contours, were designated on the map by boxed numerals, and the head loss in each intervening main was computed as the difference between the hydraulic elevations at the two ends of each line. All loops were numbered in sequence (roman numerals), and the pipelines in each loop were numbered consecutively in a clockwise direction. Adjacent pairs of loops share a common pipeline designated by a single numeral.

Normal water consumption was designated by a number of concentrated output loads placed at intersection points, the total of which equaled the ordinary demand for which the system was to be designed. A single concentrated fire load was similarly designated at the most critical point in the system. The total input then was made to equal the total output.

An initial distribution of the quantity of water flowing in each pipeline in the network was assumed. Care was taken that the sum of the inflow at each junction equaled the sum of the outflow. The direction of flow in each pipeline always must be from the higher toward the lower hydraulic elevation. As the method is self-correcting, the accuracy of the first assumption of distribution is not important. The better the engineer's judgment, however, the fewer the iterations required to achieve a satisfactory balance.

A computation sheet was prepared as shown in Table 1. Columns 1 and 2 identify the loops and pipelines. For an interior pipeline common to two loops, the pipeline number is marked with an asterisk for the first loop in which it appears and marked with a dagger for the second loop in which it appears. These markings help identify interior pipelines during subsequent computations. Columns 3 and 4 contain the values of H and Q previously determined.

TABLE 2

Conversion Factors for Commercially Available Pipe Sizes

Diameter of Pipe *in.*	Conversion Factors for Equivalent Length of 8-in. Pipe
4	29.0
6	4.06
8	1.00
10	0.34
12	0.14
16	0.034
20	0.012
24	0.0048

Column 5 in Table 1 may be calculated from Eq 3; Column 6 is self-explanatory. A nomograph prepared by O'Connor (Fig. 6, to be discussed later) is helpful in making entries in Columns 5 and 6 without directly computing them from Eq 3 and the relationship $1.85 \frac{L_e}{Q}$, respectively. As stated previously, each L_e in Column 5 has the same sign as the corresponding Q

in Column 4; consequently all entries in Column 6 are positive in sign.

With the use of Eq 10, the correction factor, ΔQ, for each loop was computed and entered in Column 7 and at the foot of the group of pipelines comprising each loop. Corrections were applied in multiples of 10 gpm, to simplify the calculations, as shown in Column 7 opposite the individual pipelines in each loop. The ΔQ calculated for each loop must be added algebraically to the flow in each pipeline in that loop, but subtracted algebraically from the flow in an interior pipeline common to another loop when that loop is studied. Thus, each common pipeline may have two correction factors in Column 7 in the loop in which it first appears, the second factor being carried backward from the subsequent loop and given the opposite sign from that appearing in the latter loop.

The corrected flows were entered in Column 8. They consist of the algebraic sum of the previous flow in each pipeline (Column 4) and all corrections appearing in Column 7. For each interior pipeline that appears in two loops, entry of the flow may be omitted in the second loop in order to avoid duplication in computing these flows and possible error. Computation of successive sets of correction factors and corrected flows may be repeated as many times as necessary to balance the equivalent lengths of all pipelines in each and every loop to a practical degree of accuracy. In this example, two corrections were made to find that the magnitude of all ΔQ's was less than 10 gpm. The system then was considered balanced, because correction fac-

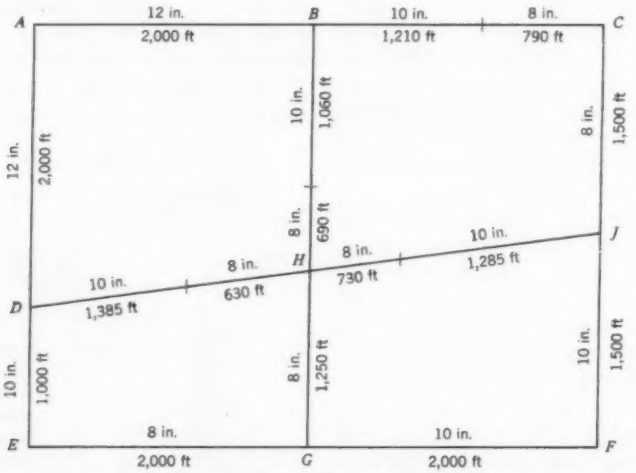

Fig. 2. Diameters and Lengths of Pipe in System in Example 1

The total length and weight of each size of cast-iron pipe were: 12-in. pipe: 4,000 ft, 127 tons; 10-in. pipe: 9,440 ft, 231 tons; and 8-in. pipe: 7,590 ft, 140 tons. This amounted to the overall total length and weight of 21,030 ft and 498 tons (compare data for Fig. 3).

tors of this size, or less, were considered negligible in comparison to the magnitude of the flow in any pipeline in the network.

After the equivalent lengths had been computed, the actual lengths were measured on the map and entered in Column 10 (Table 1). On the assumption that C was equal to 100 (Column 11) for the entire system, the sizes of mains were computed from

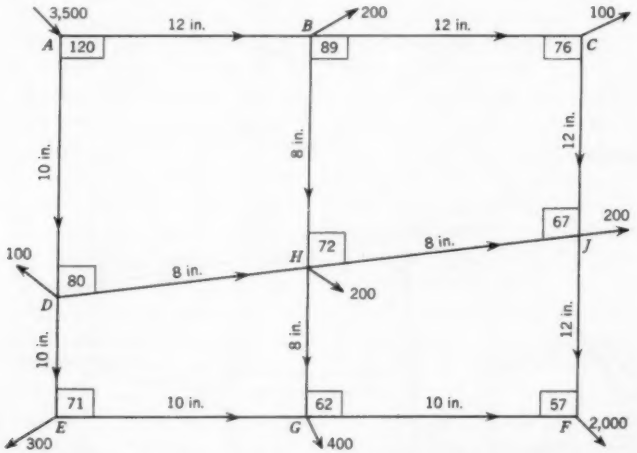

Fig. 3. Alternate Solution to System in Example 1 by Conventional Method

Boxed numerals represent hydraulic elevations. Numerals adjacent to arrows represent flows in gallons per minute. The total length and weight of each size of cast-iron pipe were: 12-in. pipe: 7,000 ft, 222 tons; 10-in. pipe: 7,000 ft, 172 tons; and 8-in. pipe: 7,030 ft, 130 tons. This amounted to the overall total length and weight of 21,030 ft and 524 tons (compare data for Fig. 2).

Eq 2 or obtained from the nomograph in Fig. 7. The computed diameters usually are odd sizes. Hence, the nearest commercially available size should be used for the entire length of each line, or Table 2 may be used to divide the total length into two partial lengths of two commercially available pipe diameters.

For example, Pipeline 2 had an actual length (L) of 1,750 ft (Column 10, Table 1) and an equivalent length (L_e) of 1,050 ft (Column 9). From Fig. 7, D was found to be 8.9 in. (Column 12). If S represents the partial length of 10-in. pipe to be used, and $1{,}750 - S$, the partial length of 8-in. pipe, the following equation may be written:

$$0.34S + 1.0(1{,}750 - S) = 1{,}050.$$

After the above equation was solved, it was found that the required length of 1,750 ft of 8.9-in. pipe could be converted to 1,060 ft of 10-in. pipe plus 690 ft of 8-in. pipe.

Figure 2 shows the final pipe sizes and lengths selected for the system discussed in Example 1 (Fig. 1).

Figure 3 illustrates an alternate solution for this problem by the conventional trial-and-error technique of assuming pipe sizes and then finding

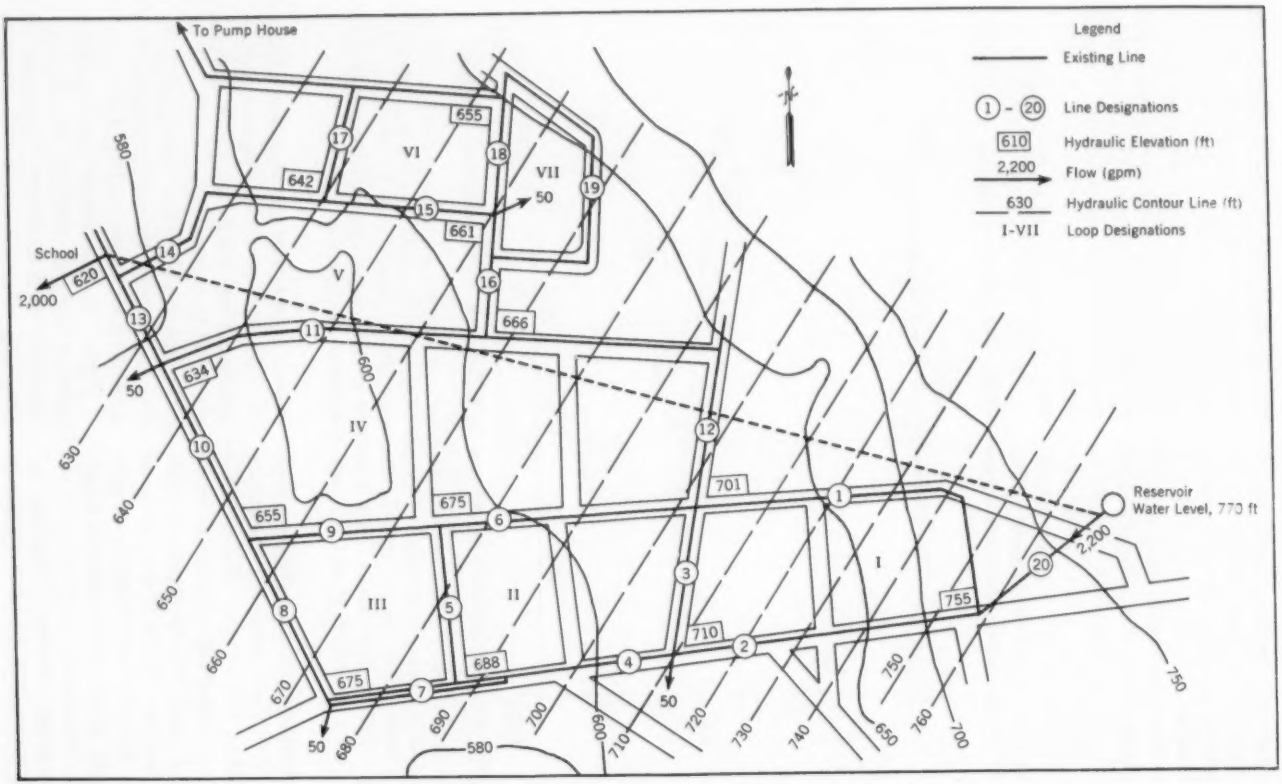

Fig. 4. Existing System in Example 2

The dashed curve represents the maximum slope of the hydraulic gradient between the reservoir and school.

the head losses in the system by the Hardy Cross method. This solution is based on the identical overall total pipe lengths given and used in the first solution (Fig. 2). But except where the fire demand occurs, at F, the pressures throughout the system are lower than those in the first solution (Fig. 1). The data in the note to Fig. 2 should be compared with the data in the note to Fig. 3.

Example 2. A study of an existing water distribution system for a village of 2,000 people was designated Example 2. An investigation was made to determine the immediate improvements to the system that would be needed to provide a fire flow of 2,000 gpm at a school location on the westerly side of the village (Fig. 4), with a minimum residual pressure requirement of 20 psi (46 ft of head) throughout most of the system. A 450,000-gal storage reservoir at the easterly side of the village was assumed to be the only source of water feeding the system under these conditions. The average friction coefficient, C, was found to be 100 for the existing pipelines, and assumed to be 120 for new pipe used for improvements.

Figure 4 is a plan of the village, showing streets, water mains, reservoir, and topographic contour lines. The lengths and diameters of the water mains are listed in Columns 3 and 4 of Table 3. The hydraulic elevation was found to be 770 ft at the source of supply (reservoir). It was required to be at least 620 ft at the point of maximum fire flow, in order to maintain the minimum residual pressure there. By use of the shortest route between the source and the fire location as a reference line, hydraulic contours at 10-ft intervals were constructed by a series of parallel lines, as shown by the dashed lines in Fig. 4. This set of contour lines provided reasonably even distribution of head losses along all routes and maintained residual pressures of 20 psi, or higher, in most parts of the village, with the exception of the northerly edge of the network. The hydraulic elevation at the upper end of Line 18 was taken to be 655 ft; the ground elevation also was approximately 655 ft. In order to raise the residual pressure to 46 ft of head at this point, an extensive pipeline network would have to be built around this area. In the interest of overall economy to the village, the minimum pressure requirement in this area during maximum fire draft was sacrificed for the present. Moreover, it was considered that the well pumps, which are the village's ultimate source of water and which feed into the network from the north, would be able to sustain some small residual pressure in this area. For this hydraulic study, however, the pumps had been assumed to be turned off or inoperative.

The hydraulic elevations at all intersections were determined from the hydraulic contour lines and marked in Fig. 4. Head losses were computed for all lines and entered in Column 5 of Table 3. Then a check of the head losses throughout the system was made mathematically to insure that Eq 4, $\Sigma H = 0$, was satisfied in each loop before the subsequent calculations were carried out. By use of the same iterative procedure described in Example 1, the equivalent lengths of all pipelines were obtained, after three corrections. By means of Fig. 7, the required diameters of all pipelines were entered in Column 10 of Table 3.

The computed diameters were compared with the actual diameters of the mains in the existing system. Where

the sizes of the existing pipelines were found smaller than required, they were marked with a double dagger (‡) in Column 10 of Table 3. Each such line, presumably, would need to be improved, if the system were to be brought into full conformity with the computed solution. A scheme of reinforcing was proposed, as shown in Fig. 5. This scheme was modified slightly from the computed solution, in order to have a more practical arrangement consistent with actual field construction conditions.

TABLE 3

Computation Sheet for Example 2

1	2	3	4	5	6 (Assumed Flows)				7 (1st Correction)			
Loop	Pipe-line	L ft	D in.	H ft	Q gpm	L_e ft	$1.85\frac{L_e}{Q}$	ΔQ gpm	Q gpm	L_e ft	$1.85\frac{L_e}{Q}$	ΔQ gpm
I	1	1,750	4	−54	−900	−2,250	4.6	−20	−920	−2,120	4.3	0
	2	1,520	8	45	1,300	940	1.3	−20	1,280	980	1.4	0
	3*	470	4	9	480	1,180	4.5	−20+20	480	1,180	4.5	0+20
				0		−130	10.4	= −13		40	10.2	=4
II	3†			−9		−1,180	4.5	−20		−1,180	4.5	−20
	4	700	‡	22	770	1,220	2.9	−20	750	1,260	3.1	−20
	5*	500	4	13	300	4,100	25.0	−20+10	290	4,400	28.0	−20+10
	6*	830	4	−26	−390	−5,000	24.0	−20+40	−370	−5,650	28.0	−20+20
				0		−860	56.4	= −15		−1,170	63.6	= −19
III	5†			−13		−4,100	25.0	−10		−4,400	28.0	−10
	7	440	§	13	470	1,750	7.0	−10	460	1,850	7.5	−10
	8	600	4	20	420	3,400	15.0	−10	410	3,600	15.5	−10
	9*	650	4	−20	−690	−1,360	3.6	−10+40	−660	−1,460	4.1	−10+20
				0		−310	50.6	= −6		−410	55.1	= −7
IV	6†			26		5,000	24.0	−40		5,650	28.0	−20
	9†			20		1,360	3.6	−40		1,460	4.1	−20
	10	620	4	21	1,110	540	0.9	−40	1,070	600	1.0	−20
	11*	1,100	4	−32	−330	−8,600	48.0	−40+20	350	−7,600	39.0	−20+20
	12	1,250	4	−35	−990	−1,220	2.5	−40	−1,030	−1,100	2.0	−20
				0		−2,920	79.0	= −37		−990	74.1	= −13
V	11†			32		8,600	48.0	−20		7,600	39.0	−20
	13	380	‖	14	1,390	260	0.3	−20	1,370	260	0.4	−20
	14	850	6	−22	−610	−1,850	5.7	−20	−630	−1,750	5.2	−20
	15*	550	6	−19	−250	−8,300	60.0	−20+0	−270	−7,400	51.0	−20+10
	16	280	6	−5	−660	−360	1.0	−20	−680	−340	0.9	−20
				0		−1,650	115	= −14		−1,630	96.5	= −17
VI	15†			19		8,300	60.0	0		7,400	51.0	−10
	17	860	6	−13	−360	−2,900	15.0	0	−360	−2,900	15.0	−10
	18*	470	6	−6	−180	−4,900	50.0	0+0	−180	−4,900	50.0	−10+0
				0		500	125	=4		−400	116	= −3
VII	18†			6		4,900	50.0	0		4,900	50.0	0
	19	1,050	6	−6	−180	−4,900	50.0	0	−180	−4,900	50.0	0
				0		0	100	0		0	100	0
Line to Reservoir												
	20	620	8	15	2,200	120						

* Interior pipeline common to two loops, in the first loop in which it appears.
† Interior pipeline common to two loops, in the second loop in which it appears.
‡ For 350 ft of 8-in. pipeline, plus 350 ft of 6-in. and 4-in. parallel pipelines.
§ For 6-in. and 4-in. parallel pipelines.
‖ For 8-in. and 4-in. parallel pipelines.

A final computation on the improved system, with the use of the conventional Hardy Cross method of balancing head losses in a system of fixed pipe lengths and diameters, was made to check the results against the initial requirements. Final hydraulic elevations at various points are marked in Fig. 5, in order to compare them with the originally established hydraulic elevations shown in Fig. 4. The proposed scheme of improvements was generally satisfactory, yielding residual pressures generally equal to, or greater

TABLE 3—*Computation Sheet for Example 2 (contd.)*

1	2	8 (2nd Correction)				9 (3rd Correction)				10	11
Loop	Pipe-line	Q gpm	L_e ft	$1.85\frac{L_e}{Q}$	ΔQ gpm	Q gpm	L_e ft	$1.85\frac{L_e}{Q}$	ΔQ gpm	Required D ($C=100$) in.	Diameter of New Lines Added to System in.
I	1	−920	−2,120	4.3	0	−920	−2,120	4.2		7.6‡	8
	2	1,280	980	1.4	0	1,280	980	1.4		8.7	
	3*	500	1,100	4.1	+10	510	1,060	4.0		6.7‡	
			−40	9.8	= −4		−80	9.6	= −8		
II	3†		−1,100	4.1	−10		−1,060	4.0			
	4	730	1,350	3.3	−10	720	1,400	3.5		6.9	
	5*	280	4,700	31.0	−10+10	280	4,700	31.0		5.1‡	
	6*	−370	−5,650	28.0	−10+15	−365	−5,700	27.0		5.4‡	8 & 6
			−700	66.4	= −11		−660	65.5	= −10		
III	5†		−4,700	31.0	−10		−4,700	31.0			
	7	450	1,900	8.0	−10	440	2,000	8.5		5.9	
	8	400	3,750	17.0	−10	390	3,900	18.0		5.5‡	6
	9*	−650	−1,500	4.3	−10+15	−645	−1,520	4.3		6.6‡	6
			−550	50.3	= −11		−320	61.8	= −5		
IV	6†		5,650	28.0	−15		5,700	27.0			
	9†		1,500	4.3	−15		1,520	4.3		7.9‡	8
	10	1,050	650	1.1	−15	1,035	660	1.2		5.4‡	
	11*	−350	−7,600	39.0	−15+10	−355	−7,400	38.0		8.2‡	
	12	−1,050	−1,090	1.9	−15	−1,065	−1,050	1.8			8§
			−890	74.3	= −12		−570	72.3	= −8		
V	11†		7,600	39.0	−10		7,400	38.0			
	13	1,350	270	0.4	−10	1,340	280	0.4		8.5	
	14	−650	−1,680	4.8	−10	−660	−1,600	4.4		7.0‡	
	15*	−280	−6,900	44.0	−10+0	−290	−6,400	41.0		4.9	
	16	−700	−330	0.9	−10	−710	−320	0.8		7.7‡	
			−1,040	89.1	= −12		−640	84.6	= −7		
VI	15†		6,900	44.0	0		6,400	41.0			
	17	−370	−2,700	14.0	0	−370	−2,700	14.0		6.3‡	6§
	18*	−190	−4,450	47.0	0+5	−185	−4,700	47.0		5.0	
			−250	105.0	= −2		−1,000	102.0	= −10		
VII	18†		4,450	47.0	−5		4,700	47.0			
	19	−180	−4,900	50.0	−5	−185	−4,700	47.0		5.9	
			−450	97.0	= −5		0	94.0	0		
Line to Reservoir											
	20									11.2‡	8§

* Interior pipeline common to two loops, in the first loop in which it appears.
† Interior pipeline common to two loops, in the second loop in which it appears.
‡ Pipelines requiring improvement.
§ Parallel to existing pipeline 17, but at another location.

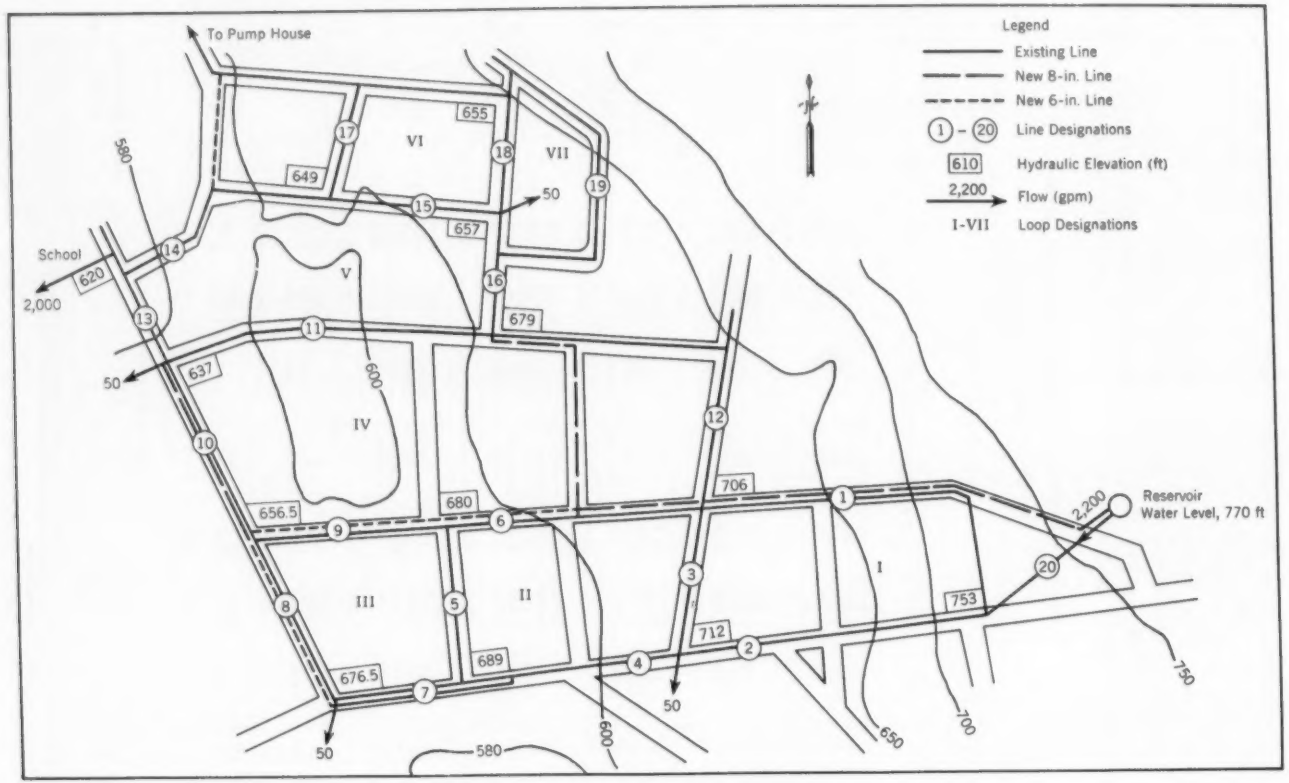

Fig. 5. Improved System in Example 2

than, those established as being acceptable in Fig. 4.

In a more complex layout, it may be necessary to make analyses for the maximum fire demand at several points. The results of the fire flow analysis of one area might require in some places larger pipe sizes than those determined from the fire flow analysis for other areas in the same system. In this event, the several results would be compared to establish the pipe sizes for the optimum practical solution.

Use of Nomographs

Two nomographs simplify the computations needed in this new method of network analysis. The nomograph shown in Fig. 6 yields solutions of Eq 3 and the right-hand side of Eq 9 simultaneously. A straight line drawn through values of H and Q on their respective scales will intersect the scales for $1.85\frac{L_e}{Q}$ and L_e at the corresponding values for these functions. For any fixed value of Q, the values of H, L_e, and $1.85\frac{L_e}{Q}$ in this nomograph are directly proportional to each other. When L_e is too large or too small for its scale, the results for its corresponding H and $1.85\frac{L_e}{Q}$ can be found by use of values proportional to n times the actual L_e on the chart, and then by division of each of the results from the nomograph by n.

The second scale, or Q scale, of Fig. 6 can be used also for determining the actual length of pipeline to solve Eq 2 when C equals 100 only. A straight edge connecting the actual length on the second scale to the equivalent 8-in. length (L_e) on the fifth scale will intersect the actual diameter (D), in inches, given on the fourth scale.

The nomograph shown in Fig. 7 was designed to solve Eq 2 for the general case [3] (when C does not equal 100). The nomograph yields any one of the four values of C, D, L, and L_e when the other three are given. The use of this nomograph may be illustrated by two problems.

Problem 1: Find L_e, given the other three factors. Connect the actual length (L) of 2,000 ft and the diameter (D) of 12 in. with a straight line, intersecting the pivot axis at Point 0_1. Connect Point 0_1 and the friction factor (C) of 120 with another straight line intersecting the length scale. Read the equivalent length (L_e) as 205 ft.

Problem 2: Find D, given the other three factors. With a straight line, join the equivalent length (L_e) of 840 ft with the friction factor (C) of 100, intersecting the pivot axis at Point 0_2. With another straight line, join Point 0_2 and the actual length (L) of 2,000 ft. Read the answer, 9.5 in., at the intersection with the D scale.

Because L and L_e are both plotted on Scale 1, the following rule should be observed when Fig. 7 is used: a line connecting a point on the pivot axis to a point on the D scale always gives the actual length (L) on the length scale; a line connecting a point on the pivot axis and a point on the C scale always gives the equivalent length (L_e) on the length scale.

Figure 7 is also useful in determining the equivalent length of a number of pipelines carrying water in series in a network. The equivalent lengths of the several component pipelines, as determined from Fig. 7, are added together to obtain the overall equivalency. Figures 6 and 7, together, are

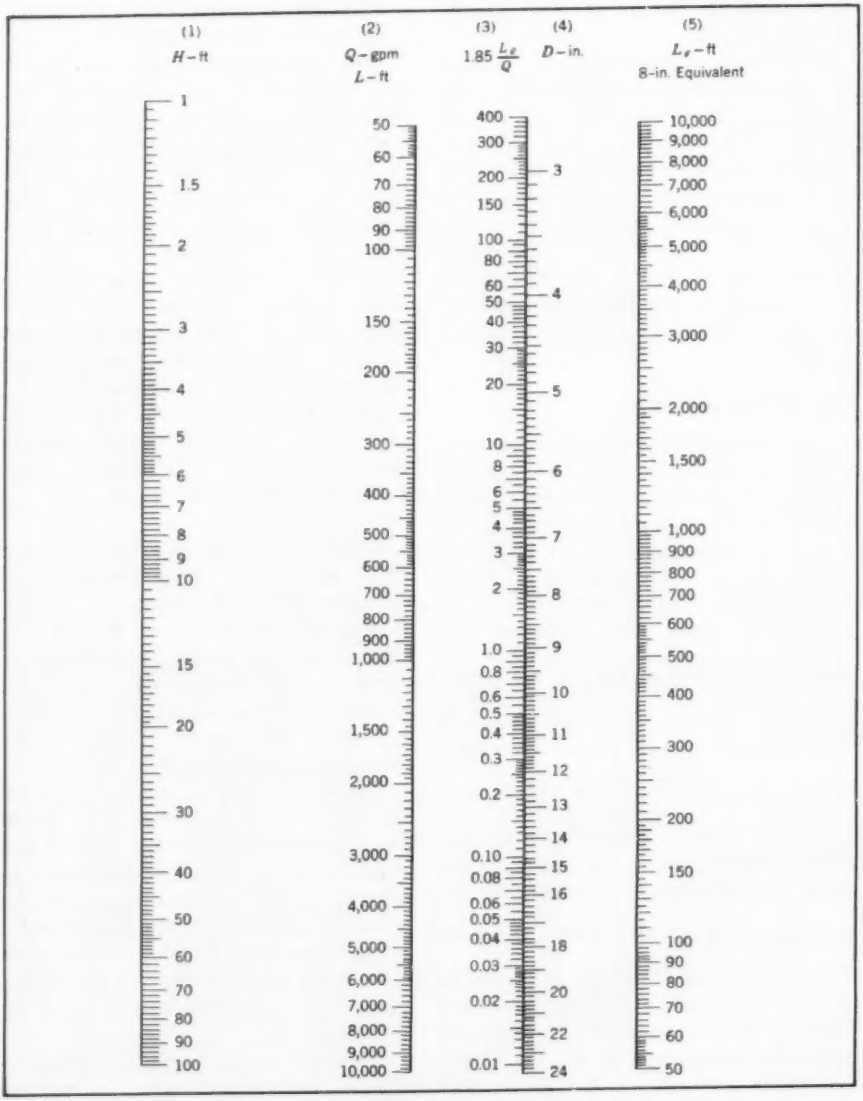

Fig. 6. Nomograph for Solution of Equations 3 and 9

A straight line drawn through values of H *and* Q *on their respective scales will intersect the scales for* $1.85 \frac{L_e}{Q}$ *and* L_e *at the corresponding values for these functions. See text under "Use of Nomographs" for complete instructions.*

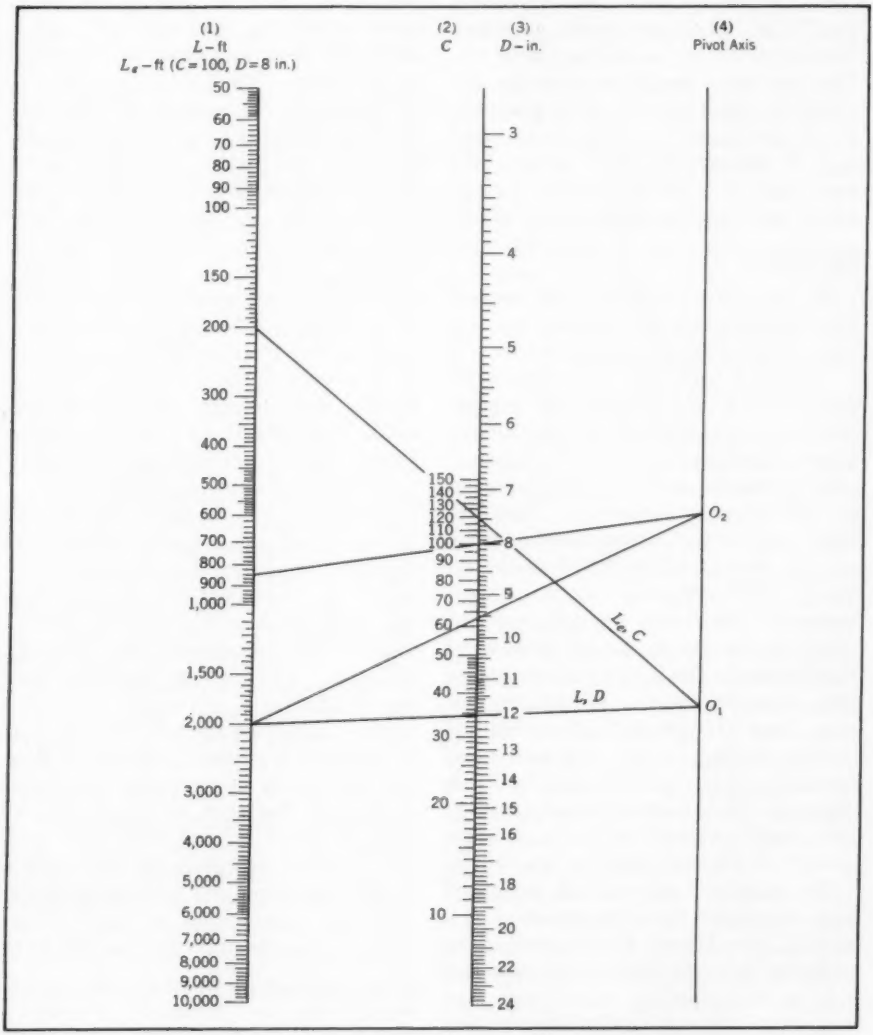

Fig. 7. Nomograph for Solution of Equation 2

The nomograph yields any one of the four values C, D, L, *and* L_e *when the other three are given. The equation pertinent to this nomograph is:*

$$L_e \doteq \left(\frac{100}{C}\right)^{1.85} \left(\frac{8}{D}\right)^{4.86} L.$$

See Problems 1 and 2 under "Use of Nomographs" for complete instructions.

useful in determining the equivalent length of two or more pipelines carrying water in parallel in a network. The equivalent length of each pipeline is first found from Fig. 7. Then the H, Q, and L_e scales of Fig. 6 are used to find the one L_e that permits the same total Q as in the parallel components, all for the same value of H.

Discussion

Hardy Cross method. The method presented here for the analysis and design of water distribution networks is a relaxation method, very similar to that of Hardy Cross. But the original Hardy Cross method and its subsequent improvements are all trial-and-error methods, insofar as the selection of pipe sizes is concerned. The designer has to try various combinations of sizes, then check their hydraulic adequacy. The successive corrections inherent in relaxation methods are applied, in the Hardy Cross method, to the hydraulic check, not to the successive improvement of the selection of pipe sizes. If the original estimate of sizes is wrong, a new selection must be made and the whole hydraulic check repeated. It is sometimes considerably less than obvious which pipe sizes should be changed, and by how much.

By contrast, the method presented here eliminates the trial-and-error feature of the Hardy Cross method by bringing the relaxation correction factors to bear directly on relative pipe resistances, or equivalent lengths, that are related directly to pipe sizes. Other than as a final verification of the pipe sizes selected directly from the results of this analysis, no hydraulic checks are necessary. Accordingly, this method is particularly useful in the design of wholly new distribution networks. It is also extremely useful in locating the mains requiring reinforcement in existing networks and evaluating the sizes necessary for such reinforcements. The latter application is performed by an analysis of the network as though it were to be a wholly new one, followed by a comparison of the resultant sizes with the actual ones. Although the new method is especially applicable to the foregoing types of network analyses, it cannot supplant the Hardy Cross method in importance in the determination of C values, based on field flow tests of existing systems,[4] or in checking the hydraulic effect of specific improvements on existing systems, especially if such improvements must be limited to partial improvements because of financial considerations.

Concept of new method. The key to the new method, of course, is the concept of balancing relative pipe resistances, or equivalent lengths, in each and every loop in the network, as expressed mathematically by Eq 5: $\Sigma L_e = 0$. The authors reasoned that the techniques developed in this method, including the use of Eq 5, tend to establish a general evenness of flow and uniformity of stressing of the network, and that such conditions should tend to result in an economical system. Some indication of the validity of this reasoning may be seen, perhaps, after an examination of Eq 2, from which it may be observed that the ratio $\frac{L}{L_e}$ is proportional to $D^{4.86}$. This equation is vital for translating the various equivalent lengths (L_e) of 8-in. pipe into actual lengths (L) of various actual sizes of pipe (D). Unless topography and usage distort L_e to an unusually small value, or unless the geometry of the system distorts L to an unusually large value, the fact that it

is the 4.86 power of D which is proportional to the ratio $\frac{L}{L_e}$ probably can be depended on to suppress any gross distortion of pipe sizes in the interest of the evenness of flow and uniformity of stressing of the system which result from the use of Eq 5. Furthermore, this same proportionality acts to retain the evenness of flow when it becomes necessary to use standard pipe sizes rather than theoretic ones. The authors' experience indicates that, under practical circumstances, the total weight of pipe necessary in a typical system designed by this method is at a minimum.

Use of computers. For small water networks, it is relatively easy to compute flow distributions manually with the aid of the nomographs presented (Fig. 6 and 7). For large systems, the electronic digital computer has proved to be a very valuable tool for calculating flow distributions by the method of balancing equivalent lengths. The previously mentioned trial-and-error nature of pipe size selection complicates the use of digital computers in the design of networks by the Hardy Cross method, especially if the engineer must utilize a commercial or university computer facility. Either the computer stands idle for relatively long, expensive intervals, while the engineer decides what to try next, or many short periods of computer time have to be scheduled and rented over many days or weeks, in order to give the engineer time to analyze the results of one trial before proceeding to another. The lack of trial and error in the method of balancing equivalent lengths eliminates the foregoing objections to the use of rented computer facilities. It should be quite feasible even to utilize the mails to transmit problems to distant computer facilities for solution by this method.

Computer programs have been developed and used successfully by many institutions and consulting engineers to perform water distribution system analyses by the Hardy Cross method.[5] These well developed programs can be modified easily to solve water network problems by the method outlined herein. The calculations for equivalent lengths made in Table 3 have been programed and computed by the senior author on a medium-sized digital computer * in a floating-decimal interpretive coding system (Bliss system[6]). It takes about 2 sec of machine time per line per iteration to make this computation. The maximum size of the network that can be handled by a digital computer depends on the storage capacity of the computer. The computing speed also varies for different types of machines and different coding systems. In any event, modern electronic computing devices have much appeal for the handling of successive approximation procedures, such as those in the new method, especially for large systems for which a great many numbers must be subjected to repetitious calculations.

Adaptation to other formulas. It should be pointed out, perhaps, that although the authors have developed the method of balancing relative pipe resistances, or equivalent lengths, around the Hazen-Williams formula, nothing in the method itself precludes its ready adaptation to other hydraulic pipe-flow formulas, such as the Darcy-Weisbach relationship, which has a somewhat more fundamental place in

* No. 650 magnetic-drum data-processing machine, made by International Business Machines Corp., New York, N.Y.

fluid mechanics.[4] Any exponential pipe flow formula can be reduced to a general form that is analogous to Eq 3; $H = AQ^n$, for example. In this instance, Eq 3 would become $\Sigma A = 0$. It is not a particular pipe-flow formula that is important to this method of network analysis and design, but rather the concept that the algebraic sum of the relative pipe resistances in each network loop should be equal to zero.

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Stability Field Diagrams as Aids in Iron Chemistry Studies

John D. Hem

A contribution to the Journal by John D. Hem, Research Chemist, Water Resources Div., USGS, Denver, Colo. Publication of the article has been authorized by the director of USGS.

IRON is one of the most common and widely distributed elements in the earth's crust. It is a constituent of most rocks and soils. The chemical behavior of the element, however, generally favors its retention in minerals of low solubility. In natural water, therefore, dissolved iron is present in minor amounts, except when special chemical conditions exist. An understanding of the behavior of iron in natural water can be gained by a detailed consideration of the kind of chemical conditions that favor its solution. Such an understanding is important, because iron, even in minor amounts, is an undesirable impurity in water supplies.

For several years, research has been conducted by USGS on the chemistry of iron in natural water. A series of reports summarizing the work has been published.[1-4] The features of iron chemistry that have been studied are:

1. *Oxidation and reduction.* Iron occurs in nature in the trivalent (ferric) and divalent (ferrous) forms. The behavior of the two forms is different; the conversion of iron from one form to the other is relatively easy.

2. *Hydrolysis.* Both ferrous and ferric ions hydrolyze in solution. The form and behavior of the hydroxides are closely related to the pH of the system.

3. *Formation of compounds of low solubility.* Several anions that are commonly found in natural water, at or near the land surface, and in ground water can combine with ferrous or ferric iron to form nearly insoluble compounds. Ferrous carbonate and ferrous sulfide are important iron minerals whose solubility is related to pH.

4. *Formation of soluble complex ions.* Iron is capable of combining with certain organic ions and some inorganic ones to produce anion or cation complexes sufficiently stable and soluble to affect the transport of iron in surface water, soil moisture, and ground water.

5. *Iron in colloidal suspensions.* Particles of iron oxides or hydroxides in or near the colloidal size range may be carried with a considerable degree of stability in river water. Amounts present in this form in clear water, however, rarely exceed a few hundredths or tenths of a part per million.

6. *Biochemical factors.* Iron is an essential element in both plant and animal metabolism. Therefore, concentrations and forms of iron dissolved in natural water may often be influenced by plants and animals. The influence may take the form of catalysis, which increases the rate of a chemical reaction, or the biologic effect may involve the utilization of energy ob-

fluid mechanics.[7] Any exponential pipe-flow formula can be reduced to a general form that is analogous to Eq 3; $H = KQ^n$, for example. In this instance, Eq 5 would become $\Sigma K = 0$. It is not a particular pipe-flow formula that is important to this method of network analysis and design, but rather the concept that the algebraic sum of the relative pipe resistances in each network loop should be equal to zero.

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tained through a metabolic process to promote a chemical reaction involving iron.

Common iron-bearing minerals of rocks and soils include oxides, carbonates, and sulfides. Under certain conditions, any of these may be attacked by water, and enough iron may thus go into solution to be troublesome to the water user. Iron oxides, for example, are dissolved by water at a nearly neutral pH below the water table if oxygen is almost entirely absent. Under these conditions iron carbonate may also be quite soluble. The sulfide minerals containing iron, of which pyrite is probably the most common, are relatively insoluble in alkaline-reducing environments but can be dissolved easily when oxidizing conditions prevail. In the following discussion, these relationships will be considered quantitatively.

Redox Potential and Iron Behavior

The oxidation of an ion of ferrous iron is represented by:

$$Fe^{++} \rightarrow Fe^{+++} + e$$

in which e represents the electron, bearing a unit negative charge, that is released by the oxidation of the ferrous ion. In a solution containing ferrous and ferric ions, the relative proportion of oxidized and reduced forms is indicated by the effective supply of these charges. An inert metal electrode immersed in such a solution will attain an electrical potential whose magnitude depends on the ratio of ferric to ferrous ions. If a standard reference electrode is immersed in the solution, the electrical potential between it and the inert electrode can be measured. This potential is called the reduction-oxidation (redox) potential and is a measure of the relative activities of oxidized and reduced ions.

The redox potential is commonly represented by the symbol E or E_h, the h in the symbol indicating that the potential is measured with respect to the standard hydrogen electrode as zero. The standard potential, E^0, for a redox couple is the value attained when the reacting substances are present at unit effective concentration or activity. Whether the potential is considered negative or positive, however, is an arbitrary matter. Standard practice in electrochemistry is to consider increasing oxidizing potentials as increasing positive values. This general practice has been followed in applications of E_h in water chemistry, geochemistry, and biochemistry.

The free energy of a chemical substance, generally expressed in kilocalories, is the energy available from that material when the temperature remains constant. The standard free energy of formation of a chemical compound, a value commonly used in thermodynamic calculations, is the change in free energy that occurs when one formula weight (in grams) of the compound is formed from the elements at standard states (temperature 25°C and pressure 1 atmosphere). The net change in standard free energy in a chemical reaction is the difference between the total free energy of formation of the products and that of the initial substances. Free-energy values are useful indexes of chemical behavior.

Fundamental equations of chemical thermodynamics interrelate the standard free-energy change resulting from a chemical reaction, the equilibrium constant for the reaction, the amounts of the reacting substances present at equilibrium, the standard potential,

and the redox potential. The equations are:

$$\Delta F^0 = -RT \log_e K \quad \ldots\ldots (1)$$

$$E^0 = -\frac{\Delta F^0}{nf} \quad \ldots\ldots (2)$$

$$E_h = E^0 + \frac{RT}{nf} \log_e \frac{a_{ox.}}{a_{red.}} \quad \ldots\ldots (3)$$

in which ΔF^0 is the standard free-energy change in the chemical reaction (in kilocalories); R, universal gas constant (1.987 calories per degree per mole, or 8.314 joules per degree per mole); T, absolute temperature; n, number of electrons, represented by a multiple of e in the redox equation; f, Faraday constant (23,060 calories/volt, or 96,484 coulombs/gram equivalent); E^0, standard potential (in volts); E_h, redox potential; and $a_{ox.}$ and $a_{red.}$, effective concentrations of oxidized and reduced substances in the chemical system being considered. If oxidizing potentials are to be considered positive, these equations will give correct values for equilibrium conditions when the reactions are written as reductions—that is, when e appears on the left side of the chemical equation.

In many published articles dealing with relations of E_h and pH in chemical equilibria in natural water, the reactions have been written as oxidations, with e appearing on the right side of the expression. If this is done and oxidizing potentials are still to be considered positive, it is necessary to change the sign of E^0 either in Eq 2 or Eq 3. Pourbaix,[5] who pioneered the use of thermodynamics in studies of metallic corrosion and developed the stability field diagram, followed this procedure, and later writers [6] used the same procedures as those of Pourbaix. The author also has used Pourbaix's technique in describing the relationships among E_h, pH, and iron content of natural water.[2, 3] To be consistent in this article, redox reactions are written as oxidations, and the sign of E^0 in Eq 2 is changed.

The substitution of appropriate constants in Eq 1–3 and a change to logarithms to the base 10 give, at 25°C:

$$\Delta F^0 = -1.364 \log K \quad \ldots\ldots (4)$$

$$E^0 = \frac{\Delta F^0}{(23.06)\, n} \quad \ldots\ldots (5)$$

$$E_h = E^0 + \frac{0.0592}{n} \times \log \frac{a_{ox.}}{a_{red.}} \quad \ldots\ldots (6)$$

The E_h of a solution can be measured by means of a sensitive potentiometer or electrometer, such as a pH meter, connected to a platinum or other inert-metal electrode and a standard electrode immersed in the solution.

Measurement of E_h

Laboratory experiments [1] have shown that values of E_h measured in solutions in the laboratory agree reasonably well with values computed from pH measurements and iron concentrations in these solutions. Agreement of the computed E_h with measured E_h for ground water, however, has not been fully ascertained. The techniques for measurement of E_h in ground water are still in a rather early stage of development. The effect of oxygen from the air is generally the dominant factor in establishing the E_h of a natural water at or near the land surface. The large supply of oxygen available for solution as a result of the contact between air and water exerts a stabilizing effect on the E_h of the solution analogous to the stabilizing of pH by the action of a buffer system. In E_h measurement, the stabilizing effect is

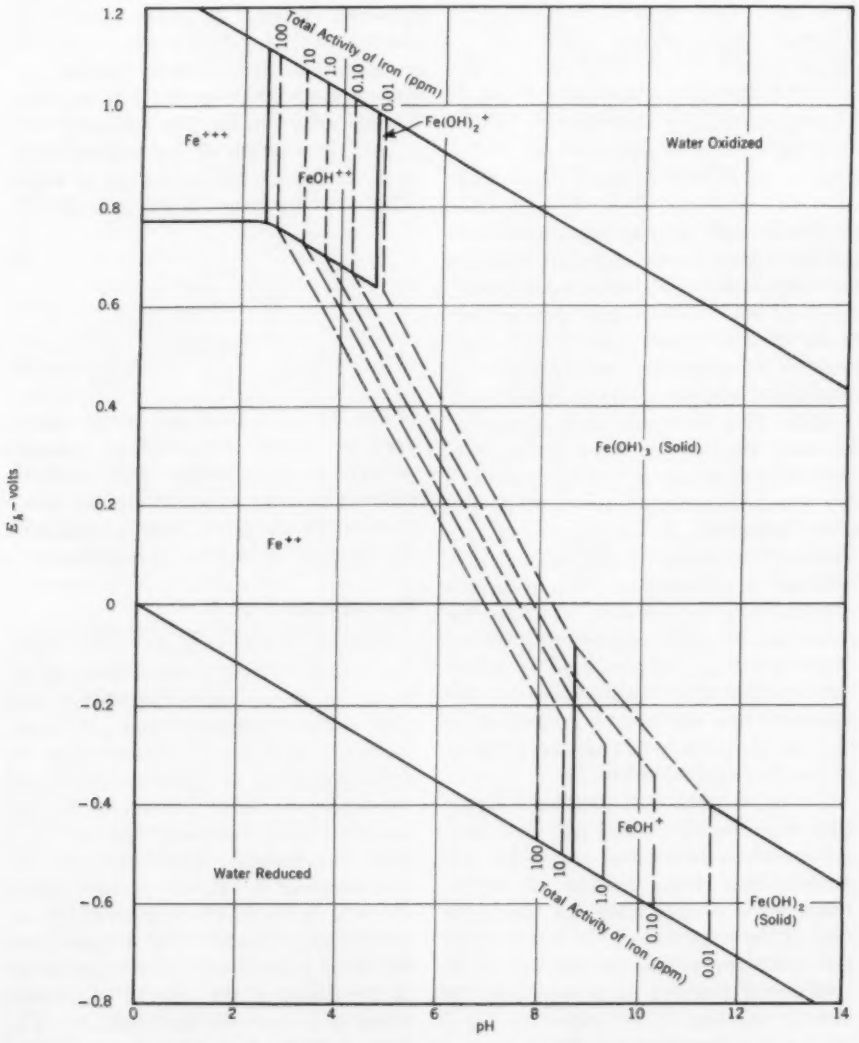

Fig. 1. Stability Fields of Ferrous and Ferric Species for Activities of 0.01–100 ppm Dissolved Iron

The upper sloping boundary represents the potential at which water is broken down by oxidation to form oxygen gas and hydrogen ions. The lower sloping boundary represents the potential at which water is broken down by reduction to form hydroxyl ions and hydrogen gas.

termed "poising." In ground water, poising effects can occur from solids that are in contact with the water. When the water is removed from the aquifer, the poising effect is diminished, because the E_h then depends only on dissolved ions that are usually present in minor amounts. The E_h then can be easily changed by introducing a new factor, such as small amounts of atmospheric oxygen. The E_h of a ground water may thus be changed during the process of pumping the water to the land surface. Some degree of E_h stratification within aquifers also could occur, the higher values occurring near the top of the water-saturated zone where some contact with air may occur.

A probing device containing E_h and pH electrodes, which does not introduce air where it is inserted, is needed for measurement of E_h in unconfined ground water. Such devices have been used for measurements in open water bodies by limnologists, and one was developed and used by the author in studies of the effects of aquatic vegetation on the iron content of water.[7] Probes have also been used for measurement of E_h in soil.[8]

Weart and Margrave [9] made measurements of E_h in water in studies of the performance of iron removal plants in Illinois. They found the measurements helpful in evaluating the efficiency of iron removal.

Since 1955, USGS has conducted experiments with methods of measuring E_h in the field, in connection with studies of the occurrence of uranium in ground water. The results of several hundred such measurements have shown that water from pumped wells usually has been affected by atmospheric oxygen by the time the water has reached the land surface where the E_h electrodes of conventional measuring equipment can be inserted in the pump discharge. A similar effect generally occurs in springs. But some measurements of E_h made by inserting the electrodes into the discharge pipe of flowing artesian wells appear to be free from the effects of oxygen.

Studies of methods of field measurement of E_h in ground water reported by Germanov and others [10] parallel the findings of USGS. The development of measuring techniques is continuing. Water in contact with the atmosphere generally has an E_h of +0.35 to 0.50 v at a nearly neutral pH.

Preparation and Use of Diagrams

The stability field diagram is a graph on which pH is plotted as the abscissa and E_h as the ordinate. The chemical system to which the diagram applies generally involves one or more solid compounds of known chemical composition in contact with water at a stated pressure and temperature. A pressure of 1 atmosphere and a temperature of 25°C are the standard conditions used. Boundaries of areas on the diagram in which particular species of ions or solids would be stable at chemical equilibrium can be computed for given concentrations of dissolved ions with the use of the equations given before.

Work by Hem and Cropper [1] showed that most hydrolysis or oxidation reactions involving iron are rapid enough for equilibrium to be attained in many natural waters, except at low pH. The departure from standard conditions of temperature and pressure in many such systems is not great enough to affect the position of the stability boundaries much. Thus, the stability field diagram is of considerable

practical value in studies of the chemistry of iron in water. Pourbaix,[5] who apparently originated the graphic procedure, has applied it extensively in his study of corrosion of metals. For example, a region of stability for metallic iron can be defined, and corrosion will not occur if E_h and pH remain at values within the field of stability for iron. During the past few years, geochemists and biochemists have used stability field diagrams rather extensively. The work of Garrels [11] is particularly noteworthy. It should be emphasized, however, that the diagram cannot be used for predicting reaction rates.

Figure 1 is a simple stability field diagram for a solution of iron in distilled water in contact with ferric hydroxide or ferrous hydroxide. Ferric hydroxide may also be considered to be hydrated ferric oxide, $Fe_2O_3 \cdot 3H_2O$. The upper sloping boundary on the diagram represents the potential at which water would be broken down by oxidation to form oxygen gas and hydrogen ions:

$$2H_2O \rightleftharpoons O_2 + 4H^+ + 4e.$$

The lower sloping boundary represents the potential at which water would be broken down by reduction to form hydroxyl ions and hydrogen gas:

$$2H_2O + 2e \rightleftharpoons H_2 + 2OH^-.$$

The locations of the water stability boundaries are determined from values for the standard potential E^0 for these two reactions and Eq 6. The standard potentials are readily available in the literature [11] or can be computed from free-energy data and Eq 5. The activities of water and the gases are unity by definition. Thus, for the oxidation reaction, Eq 6 becomes:

$$E_h = 1.23 + \frac{0.059}{4} \log [H^+]^4$$

$$= 1.23 - 0.059 \text{ pH}.$$

For the reduction reaction, Eq 6 becomes:

$$E_h = -0.83 + 0.059 (14 - \text{pH}).$$

At equilibrium, any water solution will have E_h and pH values within the limits of the water stability boundaries. Because of overvoltage effects and slowness of some reactions, however, redox potentials outside the limits can be observed. The next step in preparing Fig. 1 was to determine the positions of boundaries separating ferric species where the E_h is high enough to prevent any reduction from taking place.

Relative to this discussion, the hydrolysis reactions involving ferric ions in distilled water are:

$$Fe^{+++} + H_2O \rightleftharpoons FeOH^{++} + H^+$$

$$FeOH^{++} + H_2O \rightleftharpoons Fe(OH)_2^+ + H^+$$

$$Fe(OH)_2^+ + H_2O \rightleftharpoons Fe(OH)_3\,c + H^+.$$

The symbol *c* following a chemical formula means that the substance is present in the crystalline or solid state.

Similar reactions could be written for the various ferrous species. If the redox potential is high or low enough so that the iron is essentially all present either in the ferric or ferrous form, the species of ion that will predominate is a function of pH alone. Data in the literature relating to these reactions have been collected.[1] Equilibrium (hydrolysis) constants for these reactions can be taken directly from published values or can be computed from standard free-energy data as given by Latimer [12] and the expression:

$$\Delta F^0 = -1.364 \log K.$$

The vertical boundaries are obtained by computing the pH at which the ratio of the activities of the two species separated by the boundary has a value of unity. For example, in the first hydrolysis equilibrium, the expression for the hydrolysis constant K_h is shown to be:

$$K_h = \frac{[FeOH^{++}][H^+]}{[H_2O][Fe^{+++}]}.$$

The water is present at unit activity. Hence, all the activity terms except [H] cancel out, and the pH for the Fe^{+++}–$FeOH^{++}$ boundary is:

$$pH = -\log K_h$$

which, for this equilibrium, has a value of 2.4. Similar computations give pH values for boundaries separating the other hydroxides of ferric and ferrous iron.

The position of the boundary of stability for the two solids ferric and ferrous hydroxide depends on the value chosen for the limit of iron solubility. Activities of solids are taken as unity. Five different boundaries for stability of solids are given as dashed lines in Fig. 1. The one giving the smallest area for stability of the solids is based on a dissolved-iron activity of 0.01 ppm, the lower limit of detection by the 2,2′-bipyridine method as it is usually applied in water analysis. An activity of iron of 1.8×10^{-7} molal is equivalent to 0.01 ppm. Other boundaries are drawn for dissolved-iron activities of 0.10 ppm, 1.0 ppm, 10 ppm, and 100 ppm. Solid lines are boundaries separating dissolved species or different solid compounds and are independent of iron concentration. The position of the stability boundaries of solids depends also on the crystal form and purity of the solid.

The boundaries separating the ferric from the ferrous species are determined from the equilibria:

$$Fe^{++} \rightleftharpoons Fe^{+++} + e$$

$$Fe^{++} + H_2O \rightleftharpoons FeOH^{++} + H^+ + e$$

$$Fe^{++} + 2H_2O \rightleftharpoons Fe(OH)_2^+ + 2H^+ + e$$

$$Fe^{++} + 3H_2O \rightleftharpoons Fe(OH)_3\,c + 3H^+ + e$$

$$FeOH^+ + 2H_2O \rightleftharpoons Fe(OH)_2\,c + 2H^+ + e$$

$$Fe(OH)_2\,c + H_2O \rightleftharpoons Fe(OH)_3\,c + H^+ + e.$$

The values of E^0 for the equilibria can be computed from free-energy data compiled by Hem and Cropper [1] and from Eq 5. E_h values can then be computed from Eq 6. In this computation, the ratios of ferric to ferrous species are taken as unity, except where one or more are present as a solid. When this condition exists, the activity of the solid is taken as unity, and the computation is made by entering in the equation a selected value for the activity of dissolved iron (0.01 ppm, 0.10 ppm, 1.0 ppm, 10 ppm, or 100 ppm). The boundary between the two solids is independent of dissolved-iron activity. For example, in the equilibrium:

$$Fe^{++} + H_2O \rightleftharpoons FeOH^{++} + H^+ + e$$

ΔF^0 equals 21.08 kilocalories. From Eq 5:

$$E^0 = \frac{21.08}{23.06} = 0.91 \text{ volts.}$$

After substitution in Eq 6:

$$E_h = 0.91 + \frac{0.059}{1} \log \frac{[FeOH^{++}][H^+]}{[Fe^{++}]}.$$

This reduces to:

$$E_h = 0.91 - 0.059\ pH$$

when the ratio of the two iron species is unity. The result of the calculation of the ferrous-ferric boundary is a

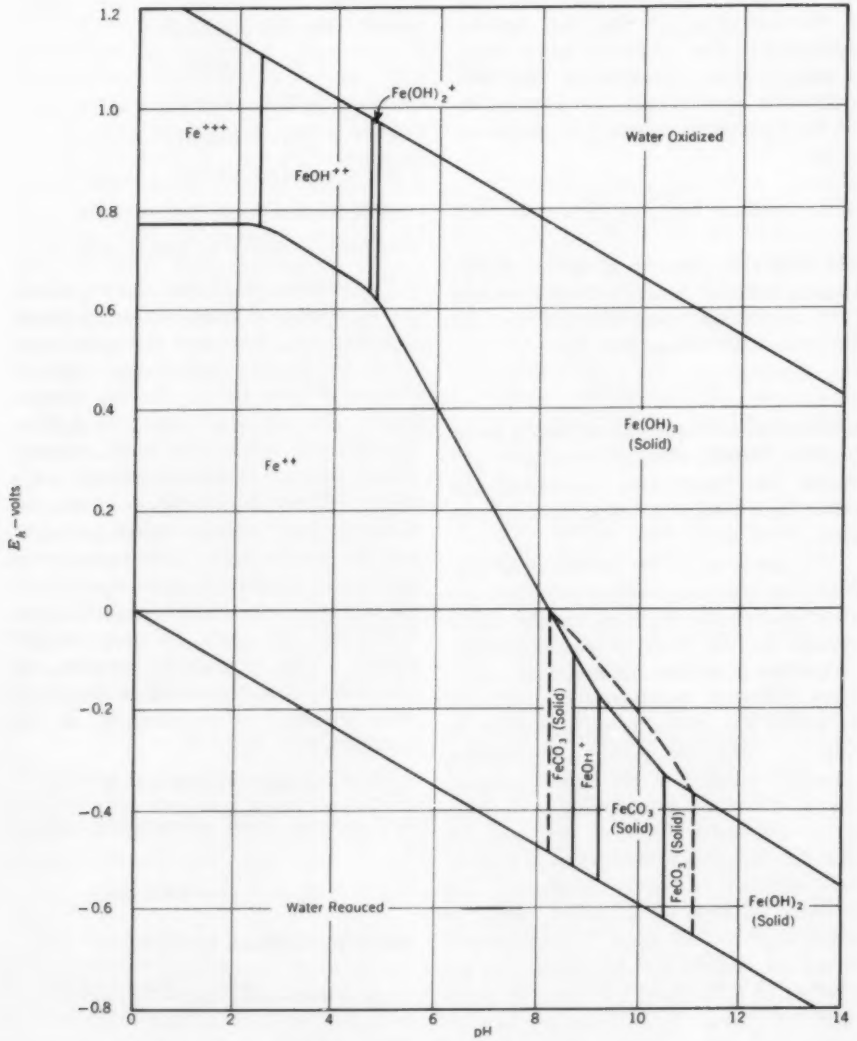

Fig. 2. Stability Fields of Ferrous and Ferric Species for Activity of 0.01 ppm Dissolved Iron

The system contains carbonate species activity equivalent to 100 ppm as bicarbonate (solid boundaries) or 1,000 ppm as bicarbonate (dashed boundaries).

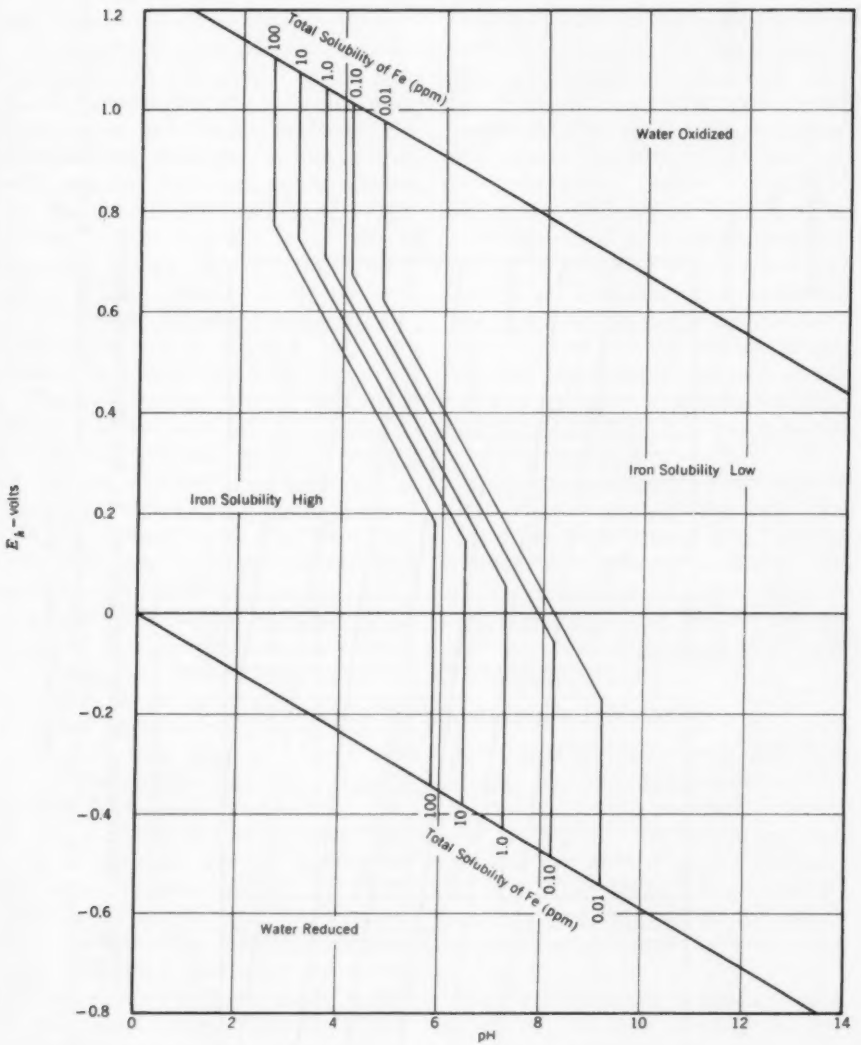

Fig. 3. Solubility of Iron in Relation to pH and E_h

The carbonate species activity was equivalent to 100 ppm as bicarbonate. Bicarbonate activity equivalent to 100 ppm or more is common in ground water.

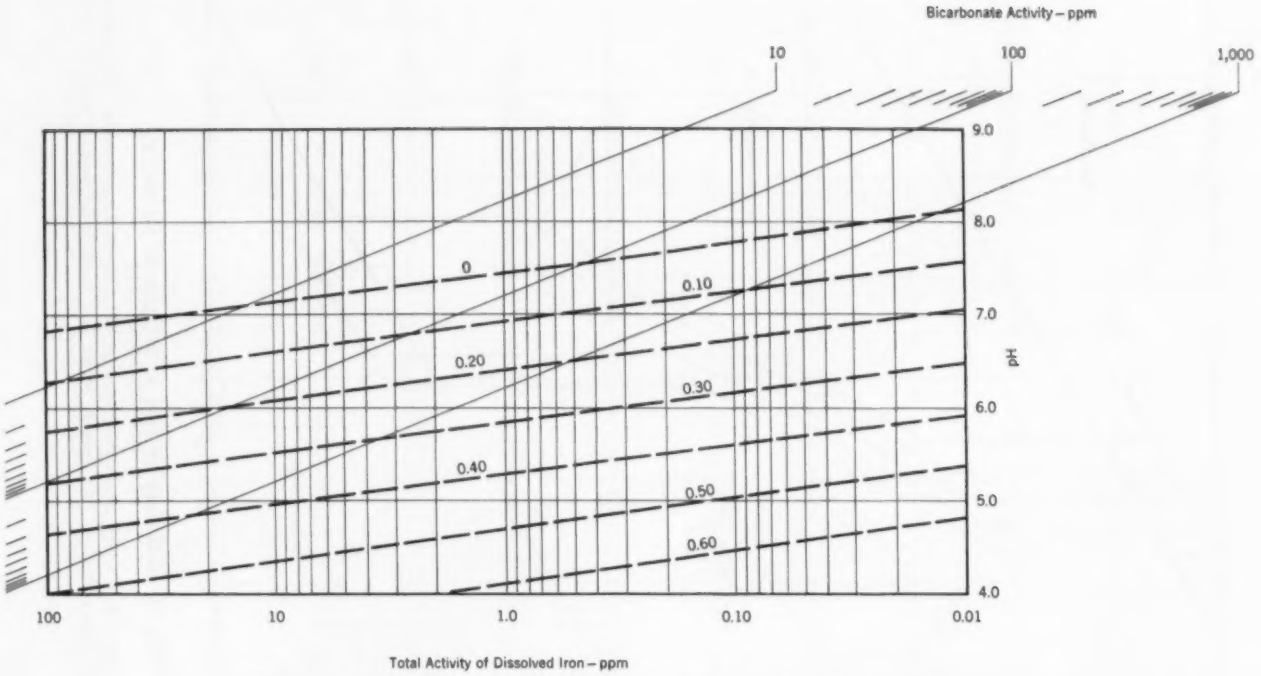

Fig. 4. Relationships of Dissolved Iron, pH, E_h, and Bicarbonate Activity

The values on the dashed curves are E_h *values. The figure represents a means of estimating* E_h *of iron-bearing ground water in its native environment.*

curve whose slope changes as it passes each of the vertical boundaries, and the form of ions or solids changes.

The stability field diagram may be applied in several ways. If the particular interest is in the solid substance involved, as in corrosion problems, the diagram will show the conditions that will minimize the attack of the solvent on the solid. In studies of water chemistry, the diagram is an aid in showing factors that affect the solubility of iron. Figure 1 can be used as a solubility diagram that depicts the correlations among E_h, pH, and iron content in a distilled-water system.

The relationships shown in Fig. 1 do not take into account certain important factors that are usually involved in natural waters. For example, if some carbon dioxide is present in solution, the presence of another solid, ferrous carbonate, will be possible and must be considered as a possible control over the solubility of iron. Solid lines in Fig. 2 represent a system containing dissolved-carbonate species, equivalent to an activity of 100 ppm in terms of bicarbonate ions, and an activity of 0.01 ppm of iron. The size of the stability region for $FeOH^+$ in Fig. 2 is much smaller than that in Fig. 1. The dashed lines in Fig. 2 show the effect of an activity of dissolved-carbonate species of 1,000 ppm in terms of bicarbonate ions. This eliminates the $FeOH^+$ field entirely. Ferrous carbonate, however, will be oxidized to ferric oxide at the E_h of aerated water; hence, it is important only in anaerobic environments.

The position of the $FeCO_3$–Fe^{++} boundary is computed from the equilibrium:

$$FeCO_3 + H^+ \rightleftharpoons Fe^{++} + HCO_3^-$$

and the relationship of the standard free-energy change to the equilibrium constant.

A three-dimensional graph could be made in which pH, E_h, and activity of carbonate species are plotted to show the effect of bicarbonate and related species over a considerable concentration range. Figure 2 represents only two bicarbonate values. Figure 3 shows the solubility of iron in water in the presence of dissolved-carbonate species in amounts equivalent to an activity of 100 ppm as bicarbonate. The figure shows that even this moderate amount of bicarbonate greatly decreases the solubility of iron at low redox potentials. Bicarbonate activity equivalent to 100 ppm or more is common in ground water.

The carbonate species considered in Fig. 2 and 3 include not only carbonate and bicarbonate ions but also undissociated carbonic acid and dissolved carbon dioxide. The undissociated forms constitute an increasingly important part of the total as the pH decreases to less than 7.

E_h of Ground Water

The relationships among pH, iron content, E_h, and bicarbonate activity are expressed in somewhat different form in Fig. 4. In the figure, iron activities are plotted as the abscissa; pH, as the ordinate. A grid of E_h curves is superimposed by the use of computations the same as were made for Fig. 1.

Figure 4 represents a means of estimating E_h of iron-bearing ground water in its native environment. The estimate can be made for any water for which there can be obtained a pH value that represents accurately the actual pH in the aquifer and which is free from suspended iron at the time of sampling. In the absence of inter-

fering ions, the iron activity for use in the E_h estimates can be obtained from the total amount of iron reported in the sample, including any iron that precipitated in the sample bottle after collection. The pH and iron values result in a point on Fig. 4 whose relationship to the E_h grid represents the E_h of the water. The figure was developed by the author during his studies of iron chemistry.[2]

An important limitation on the use of Fig. 4 is the effect of bicarbonate. In those solutions in which the bicarbonate activity is high enough to precipitate ferrous carbonate, a decrease in E_h may not have any effect in bringing more iron into solution. The curves in Fig. 3 denoting iron content are vertical in the region where iron solubility is affected by carbonate precipitation, showing that iron content is not E_h-dependent in this region. They are also vertical in the ferric region at low pH, but these conditions are not included in Fig. 4.

When Fig. 4 is used for estimating E_h, the bicarbonate activity in the solution should first be ascertained. A straight line can then be projected on the figure for the bicarbonate activity value, with the bicarbonate activity scale at the top of the diagram. The pH–iron activity intersection should then be ascertained. If this point is below the bicarbonate line for the water, the E_h can be estimated from the position of the point with respect to the E_h grid. If, however, the pH–iron activity intersection lies on the bicarbonate line or above it, the system probably is under the influence of ferrous carbonate, and an E_h estimate cannot be made. The formation of complex ions may also interfere with the use of the figure.

The difficulties of measuring E_h have been pointed out. A knowledge of the E_h in aquifers, however, is essential for studying some features of natural-water chemistry. For example, the chemical compatibility of water proposed for underground injection with water already in the formation can be predicted more readily if the E_h can be estimated. This kind of problem can be expected to occur with increasing frequency in studies of ground water recharge or pollution, or the use of deep salt water aquifers for waste disposal. In those systems for which an estimate of E_h can be made from analytic data, the method should be of practical value, even though it has limitations.

The results obtained in E_h estimates depend on the obtaining of meaningful pH determinations. The pH value obtained in the laboratory after the sample has been stored for several days or weeks and has been opened a few times, perhaps to take out portions for various chemical determinations, may have little relationship to the pH value that the water had in the aquifer underground. The only pH values that are useful in E_h estimates are those that represent the water in its natural surroundings. The pH determinations must be made in the field immediately after the water has issued from the well or spring. Some waters may change in pH even before the field determination can be made, but this degree of instability is probably unusual.

The pH measurement should be made with a method that is accurate to a pH unit of 0.1. Test paper or other devices for making pH approximations are not suitable for this purpose. The temperature of the water

should be within 10–15 deg of 25°C if Fig. 4 is to be used with reasonable accuracy.

Complex-Ion Formation

The discussion of pH-E_h-iron relationships thus far has assumed that chemical equilibrium has been established. In some natural waters, this is not a completely safe assumption. Ferrous and ferric iron form complex ions with some organic materials that may be found in soils or the leach from organic debris. The complexed iron is still subject to oxidation, but the reaction rate may be changed. Also, the E_h at equilibrium may be different from the value attained with uncomplexed iron. The author has investigated a complex involving ferrous iron and tannic acid which has an oxidation half-life in contact with air of approximately 30 days at pH values near 8.0.[3] This compares with a half-life of a few minutes for the oxidation of uncomplexed ferrous ions at this pH. Soluble complexes of ferric iron with organic matter are common. Inorganic ions such as phosphate also form ferric complexes. Also, there may be some complexing between bicarbonate and ferrous ions, but this has not been definitely ascertained. In any event, E_h-pH-iron relationships may require special study where complexing occurs. Natural water that has a noticeable color may contain complexed iron and may not yield useful data for E_h estimates.

Colloidal Ferric Hydroxide

Water may contain relatively stable colloidal suspensions of ferric hydroxide.[1] Much of the iron in clarified surface waters appears to be in this form, but the amount present generally is only a few hundredths of a part per million. The effects of colloidal iron may distort the analytic data for dissolved iron when the amount present is less than approximately 0.10 ppm. Hence, the relationships among iron, E_h, and pH may not be well defined at this concentration.

Effective and Measured Concentrations

Throughout this article, thermodynamic effective concentration or activity values are used instead of the measured concentrations obtained in water analyses. The activities of ions in dilute solutions are always somewhat less than the actual concentration present, because of interionic forces, combination with molecules of the solvent, and other effects. As total concentrations in natural waters increase, the difference between activities and measured concentrations increases also, at least up to a concentration near 5,000 ppm dissolved solids.

Methods for computing activities from concentrations are given by Klotz,[13] and some applications of these techniques to natural waters have been described by the author.[2] In waters whose concentration of dissolved solids is less than approximately 100 ppm, activities and measured concentrations will generally differ by not more than 10–15 per cent. At concentrations of approximately 1,000 ppm, however, activities of such divalent ions as Fe^{++} may be only half as great as the measured concentration values. For most waters, therefore, activity corrections of the measured concentrations are necessary to give satisfactory accuracy in calculated relationships. Measured pH values, of course, are themselves effective hydrogen ion con-

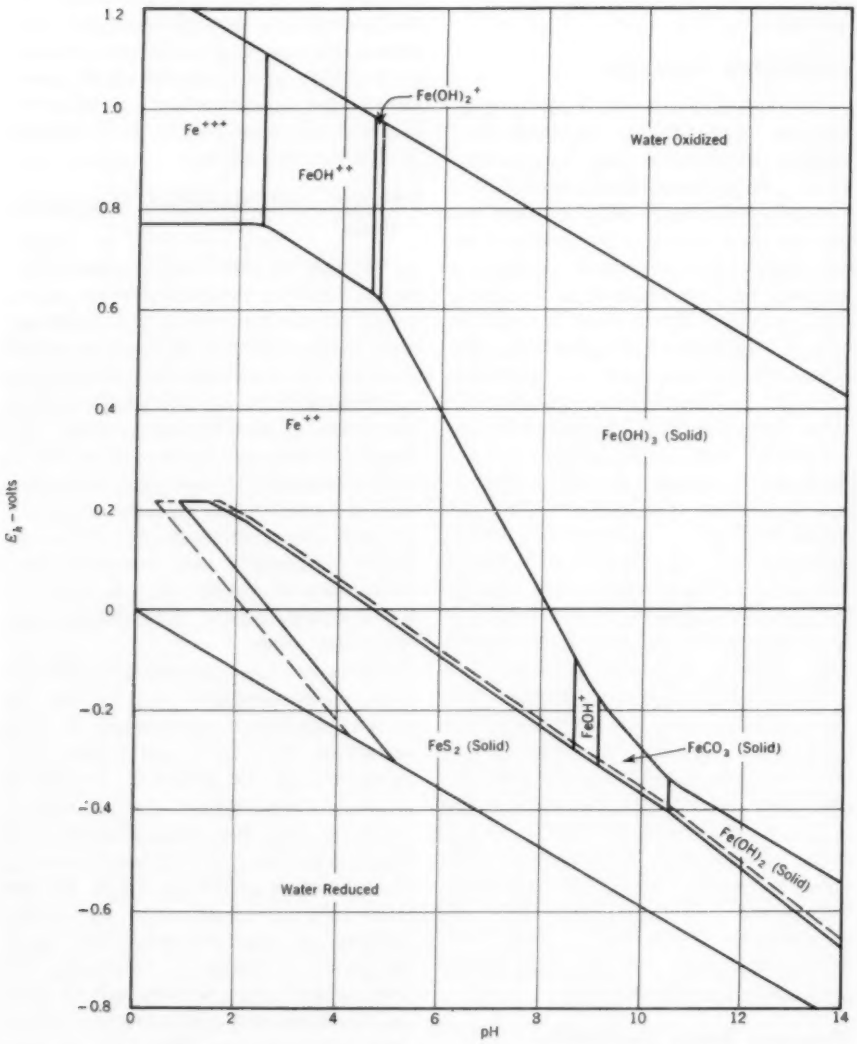

Fig. 5. Stability Fields of Ferrous and Ferric Species for Activities of 0.01 ppm Dissolved Iron and 100 ppm Carbonate Species as Bicarbonate

The system contains sulfur equivalent to 10 ppm as sulfate (solid boundaries) or 100 ppm as sulfate (dashed boundaries).

centrations, and they require no correction.

Dissolved-Sulfur Species

Sulfur is susceptible to oxidation or reduction and can exist in many forms, ranging from S^{--}, as in sulfide, to S^{6+}, as in sulfate. Thermodynamic calculations show, however, that few of these forms would be stable in natural water. A stability field diagram for sulfur can be prepared showing the forms that would be expected in water.[6] The effects of sulfur species in solution in a system that also contains iron and bicarbonate have been described in a previous study.[4] Figure 5 is a representation of such a system.

Oxidation and reduction reactions of sulfur unfortunately are not ideal subjects for representation in stability field diagrams. The equilibrium condition assumed for the preparation of these diagrams often is not attained in natural waters containing sulfate or other dissolved forms of sulfur.

The solid lines in Fig. 5 represent a system containing activities of 0.01 ppm iron, 100 pm dissolved-carbonate species as HCO_3^-, and 10 ppm dissolved-sulfur species as SO_4^{--}. The extent of the pyrite field when the sum of the sulfur species is 100 ppm as sulfate is indicated by dashed lines. The close spacing of the iron solubility lines along the top of the pyrite field shows that a minor increase in E_h could have marked effects. The solution of pyrite in this region involves oxidation of sulfur from S_2^{--} to SO_4^{--}.

Figure 6 is a solubility diagram for iron in a system at equilibrium containing a total dissolved-carbonate species of 100 ppm as HCO_3^- and a total dissolved-sulfur species of 10 ppm as SO_4^{--}. Water may retain sulfate for a very long time, even though it is subjected to E_h and pH conditions within the field of stability of FeS_2.

Bacterial Action

The effects of bacterial action on the behavior of iron are important in a number of respects. Bacteria, however, cannot produce results that are contrary to the laws of thermodynamics. They can promote such reactions as sulfate reduction, which requires an outside supply of energy, only when they are able to obtain energy from sunlight or from the breakdown of compounds that yield energy, such as in the oxidation of hydrocarbons. Bacterial action will go on if the E_h and pH of the environment are favorable for the continued existence of the bacteria. Therefore, the stability field diagrams presented here are applicable whether bacteria are present or not. The bacteria may change the rate of the reactions that would normally occur as predicted by the diagrams.

Application of Diagrams to Systems

The stability field diagrams and solubility diagrams are applicable to chemical systems that are in a state of equilibrium. In natural water in the zone of circulation in the outer part of the earth's crust, there is continual movement of water and solutes over the surface and into and out of the soil and rock materials below the surface. As a result, the pH, E_h, and dissolved-mineral content of water may change with time. Solubility diagrams, such as Fig. 3 and 6, are as applicable to a system in which water is in motion as to one in which water is at rest, if equilibrium is attained.

Solutions that have reached equilibrium with one kind of solids may be

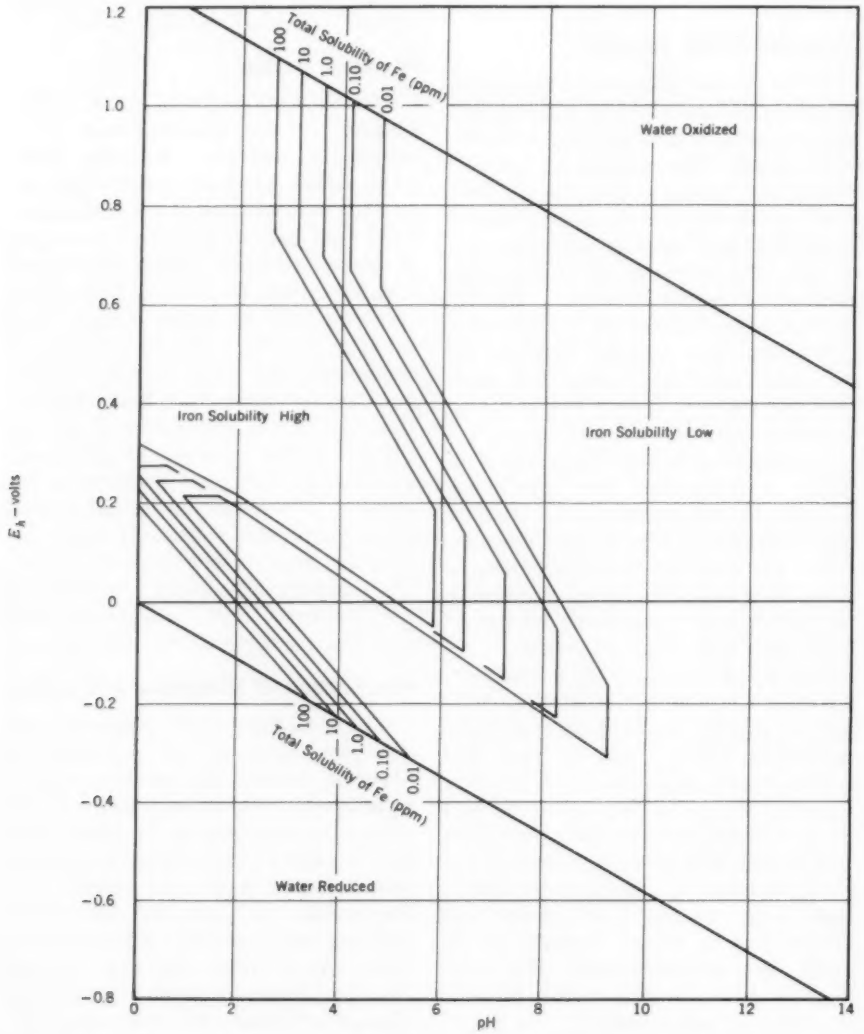

Fig. 6. Solubility of Iron in Relation to pH and E_h in a System at Equilibrium

The system contains carbonate species activity at 100 ppm as bicarbonate, and sulfur species activity at 10 ppm as sulfate.

transported into an area where the solids are entirely different, and a new set of conditions will result in changes in the composition of the solution. The E_h and pH of the solution in contact with the solid materials at a single point may change from time to time owing to the rise and fall of ground water levels and the seasonal changes of streamflow, availability of organic matter, and other factors.

Water in contact with air will have an E_h of approximately 0.35–0.50 volt, and usually its pH is greater than 5. In most of the areas relating to this condition in all the stability field diagrams, ferric hydroxide or ferric oxide is the stable form, and iron solubility is low. If the oxide is subjected to reducing conditions, however, it may be altered to give a solution containing appreciable amounts of Fe^{++}. Continued alteration will ultimately remove all the oxide, as water capable of dissolving it passes through the system and carries off the dissolved iron. The conditions could be reversed, however, by a lowering of the water table and the introduction of air, so that the dissolved iron will be redeposited in its original location or elsewhere.

Where sulfate reduction occurs, iron would be deposited as sulfide. If an iron-bearing water enters an environment in which sulfate reduction can take place, the dissolved-iron content of the moving water could easily become less than 0.01 ppm. If a pyrite deposit were subjected to oxidation, however, the close spacing of iron solubility curves, as in Fig. 6, suggests that rather large amounts of iron may be dissolved, as the difference between high and low solubility of iron at any pH is only a few hundredths of a volt in E_h. Actually, it is well known that pyrite can be readily oxidized in air when water is present and that pyrite oxidation is an important source of dissolved iron in some waters.

The numerous factors that may influence the capacity of a ground water to dissolve iron, and the ease with which some of the factors may change as water moves through the ground or as the water table rises and falls, help to explain logically some of the apparently erratic differences in iron content among similar wells and the fluctuations in iron content that sometimes occur when a well is pumped for long periods. As more information is obtained on the E_h of natural water, field evidence will become available to help show the relationships of iron and other constituents of water to the redox potential of the environment. Because such evidence is largely lacking at present, laboratory and theoretic approaches are required.

Summary

In many natural waters, the solubility of iron is controlled by the redox potential, pH, and concentrations of carbonate and sulfur anionic species. These factors can be summarized by means of stability field diagrams, which are graphs having E_h as the ordinate and pH as the abscissa. In the absence of chemical complexing and excessive amounts of carbonate, the pH and iron content of a ground water may serve as an index of the redox potential of the water in its natural environment.

Areas of stability shown on the graphs for the different ionic or solid species of iron are located by computations with the use of standard free-energy values selected from published

literature. The stability field diagrams summarize the conditions required for stability of solutions containing 0.01–100 ppm iron, and indicate some factors that control solution and precipitation of iron in natural water.

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Discussion

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The author has performed a valuable service in presenting stability field diagrams as aids for the interpretation of the chemistry of iron in natural waters. The adage that one picture is worth a thousand words is pertinent to the value of redox potential–pH diagrams. Such diagrams can quickly show how hydrogen ions and electrons simultaneously shift the various iron equilibria with varying conditions, and can indicate which species predominate under any given conditions. In the opinion of the writer, graphical methods, especially in water chemistry, can be used to much better advantage than is now realized.

A potential–pH diagram is theoretical. Naturally it contains only that information used in its construction (free-energy data for the species considered). Consequently, it presents, in most instances, an oversimplified picture, but one that leads to predictions that are generally in accord with observed behavior. The value of the diagram thus consists primarily in providing an aid for the interpretation of observed facts.

E_h Measurements in Natural Media

The object of this discussion is to caution against the indiscriminate use of E_h measurements in natural waters and to discuss some of the difficulties that, in addition to those already mentioned by Hem, should be considered in the evaluation of E_h readings. As E_h measurements can be accomplished by seemingly simple techniques, it is very tempting to use such procedures for the investigation of reduction and oxidation conditions in natural waters. One would expect that such measurements would relate, in a comprensive manner, the total oxidation or reduction intensity of a natural medium. But extensive measurements of reduction-oxidation (redox) potential in surface and ground waters and in benthal deposits have, in the writer's opinion, failed to yield results that are amenable to unambiguous interpretation. It is realized that such measurements are of great value in simple systems for which the variables are known and under control, and where equilibrium conditions are established at the electrode surface. In natural waters, these criteria are not fulfilled. Such systems are complex and involve numerous reacting components—for example, various soluble iron ions and iron complexes, dissolved oxygen, different organic redox systems, and sulfides. Few, if any, of these components are known or controlled.

Complexity of Ferrous-Ferric System

The ferrous-ferric system is generally used as the standard textbook example for introducing the concept of redox potential and for illustrating the significance of the Nernst equation. The system is so complicated, however, that anyone who attempts to verify this equation by measuring E_h in a ferrous-ferric solution soon becomes frustrated by the significant discrepancy between observed and calculated data. Numerous explanations for this discrepancy can be given. The limited solubility of ferrous and ferric iron is one important consideration. Figure 7 gives the solubilities of ferrous and ferric iron in a water containing bicarbonate. As Hem and others [1-3] have pointed out, soluble iron does not exist only as Fe^{++} and Fe^{+++} ions, but consists also of various iron-hydroxo complexes such as $FeOH^+$, $Fe(OH)_3^-$, $FeOH^{++}$, $Fe(OH)_2^+$, $Fe(OH)_4^-$, and $Fe(OH)_3$ (*d*).* Figure 7, which illustrates the equilibrium relations of some of these species, is still an oversimplification, for it does not consider such soluble polymeric species as $Fe_2(OH)_2^{4+}$ and other soluble complexes that are generally present in natural waters (such as chloro-, sulfato- and phosphato-complexes). In acid solutions, in which aqueous iron is quite soluble, "formal potentials" that have been empirically determined for the given experimental conditions must be utilized.[5] The foregoing remarks do not imply that Nernst's law is not rigorously fulfilled, but they should serve to indicate that inadequate knowledge of the actual participating species may impair the proper interpretation of E_h readings.

Redox Couples in Same System

Although redox potentials in natural waters are quite frequently related to the iron concentrations, one must not overlook the fact that numerous other reactive redox components, which are not necessarily in equilibrium with one another, may be present in natural media. It is not always possible to

* $Fe(OH)_3$ (*d*) is dissolved molecular undissociated ferric hydroxide that is believed to be in equilibrium with solid $Fe(OH)_3$.[4]

establish rigorously which of these reacting components determines or influences the measured E_h. The presence of oxygen is certainly a dominant factor in the relative "oxidation intensity" of a water. In a system where the redox potential is solely determined by the oxygen tension, a reduction in O_2 concentration from 10 mg/l to 0.1 mg/l will lower the potential by only 30 millivolts. Furthermore, the O_2-OH^- couple behaves irreversibly at a platinum electrode and thus does not give E_h values in accord with thermochemical calculations. An irreversible reaction is one in which the interchange of electrons among the electrode and the oxidized and reduced forms of the redox couple involved proceeds very slowly. Most organic redox systems behave irreversibly. Similarly, a bright platinum electrode also does not respond "properly" to the sulfide-sulfur-sulfate system.

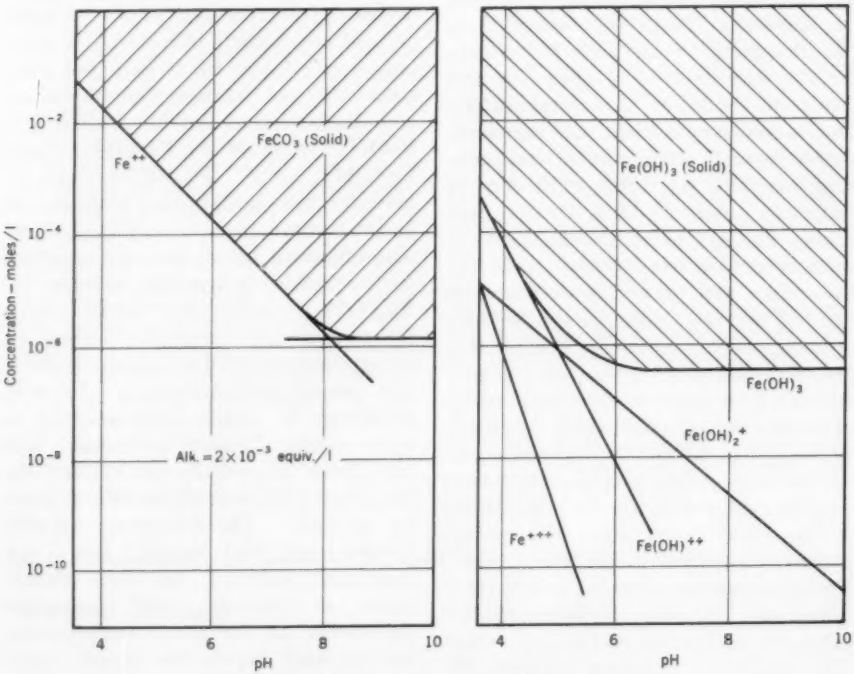

Fig. 7. Solubility of Ferrous and Ferric Iron

Left: maximum soluble ferrous iron (Fe^{++} and $FeOH^+$) in a natural water containing 100 mg/l alkalinity (as $CaCO_3$). Right: maximum soluble ferric iron (Fe^{+++}, $FeOH^{++}$, $Fe(OH)_2^+$, and $Fe(OH)_3$ (d).

It is highly questionable whether E_h should be listed as a factor in bacterial growth.[6] Certainly some organisms will not grow when there are too many reducing materials present, and some will not grow if there is too much oxidizing material. But the question of which factor is cause and which is

effect has not been clearly answered.[6] A reversible couple, if present together with an irreversible one, can act as an electron-transfer catalyst (mediator) for the irreversible couple. Thus, the dominant role that oxygen plays in establishing the E_h of a natural water is an indirect one, and it appears likely that the reversible ferrous-ferric couple is frequently the potential-determining couple. On the other hand, the ferrous-ferric couple does not always reach equilibrium with oxygen. At low pH, ferrous iron (although thermodynamically unstable) may persist in the presence of oxygen for extensive periods of time. Figure 8 summarizes some of the results obtained by Stumm and Lee[7] on the oxygenation rate of ferrous iron in an air-saturated solution. In addition to ferrous-ferric and O_2-OH^-, other reversible and irreversible redox couples may be present in natural media: NO_3^--NO_2^-, NO_2^--NH_3, sulfide-sulfur-sulfate, and Mn^{++}-MnO_2. It is thus generally not possible to relate unambiguously the measured potential to the composition of the solution.

Apparent Potentials

E_h measurements in unpoised systems such as natural waters are not simple from a technical point of view, and extreme precaution is necessary in order to obtain reproducible readings. The response of the electrode is generally very sluggish. The measuring electrode is very easily contaminated by insidious trace quantities of tensioactive materials that render potential-determining ions incapable of participating in the formation of a double layer at the electrode-solution interface. Furthermore, the Nernst equation cannot be expected to hold true at extremely low concentrations.[5]

E_h Readings as Empiric Observations

Media that contain large quantities of oxidizing agents certainly have high oxidation potentials, and those containing large quantities of reducing agents have low potentials. Empiric correlations between the distribution of iron species and the distribution of redox

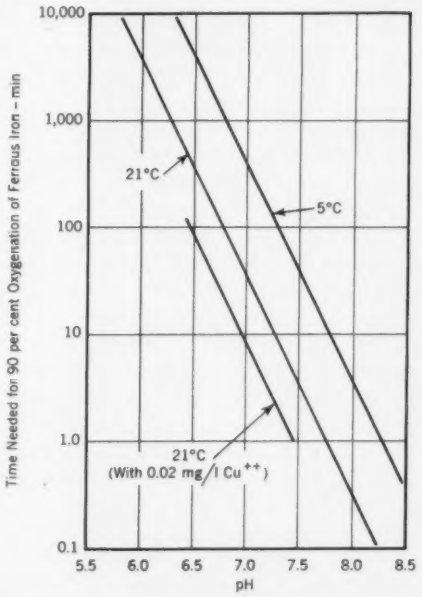

Fig. 8. Oxygenation Time as a Function of Temperature and pH

The data pertain to air-saturated HCO_3^- solutions.

potentials may sometimes be feasible. A quantitative interpretation, however, with respect to solution composition, from E_h data of natural media does not appear to be justified. In addition to their value with solutions for which the variables are known and controlled, E_h measurements may serve as valuable tools with systems that are well

poised—for example, the control of CN^- oxidation by Cl_2 or the reduction of CrO_4^{--} to $Cr(OH)_3$ in industrial wastes.

It is not the writer's intention to discourage entirely the use of redox potential measurements, but it is deemed necessary to point to some of the difficulties of measurement and interpretation. Frequently, under favorable conditions, E_h readings can supplement, but seldom entirely replace, the analytic determination of individual solution components. A trustworthy analysis of some of the pertinent constituents of the water—dissolved O_2, H^+, S^{--}, NO_2^-, NH_4^+, Fe^{++}, and Fe^{+++}, for example—is generally more informative than an E_h reading. The stability field diagram, which indicates the different equilibria that must be considered, will be a valuable tool in the interpretation of analytic findings.

The difficulties of collecting undisturbed samples for analytic purposes are appreciated. Thus, methods that involve the introduction of electrodes without contamination into the natural media are most appealing. Although the glass electrode, if used with necessary precautions, has proved to be sufficiently specific and sensitive, the platinum electrode lacks specificity. There is a pressing need to develop electrodes and monitoring systems that permit specific measurements of individual solution components *in situ*. Some progress has been made in the development of a reliable oxygen electrode,[8] and satisfactory electrodes are now available for the measurement of a few anions such as chloride and sulfide. Current research on ion-exchange membrane electrodes may lead to the development of additional specific measuring devices in the near future.

Summary

It is not intended to detract in any way from the significance of Hem's contribution on the use of stability field diagrams of iron systems. The value of potential-pH diagrams consists primarily in providing a complementary mode for the interpretation of observed facts. The measurement of E_h in natural media is plagued with many difficulties. An unambiguous interpretation of E_h readings in natural waters that contain numerous reacting redox components, which are neither known nor controlled, is not possible. There remains a well defined need for the development of specific electrodes that permit the measurement of the individual components of natural waters *in situ*.

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Effect of Fish Poisons on Water Supplies

Part 3. Field Study at Dickinson

Jesse M. Cohen, Quentin H. Pickering, Richard L. Woodward, and Willis Van Heuvelen

A contribution to the Journal by Jesse M. Cohen, Chemist; Quentin H. Pickering, Biologist; and Richard L. Woodward, Chief, Eng. Sec., all of Water Supply & Water Pollution Research Branch, Robert A. Taft San. Eng. Center, Cincinnati, Ohio; and Willis Van Heuvelen, Director of Public Health, State Dept. of Health, Bismarck, N.D.
This is the last of three articles on the effect of fish poisons on water supplies. Part 1, on removal of toxic materials, was published in the December 1960 Journal; Part 2, on odor problems, in the January 1961 Journal.

BECAUSE of the large coarse fish population in the Dickinson Reservoir, the North Dakota State Game & Fish Department proposed treatment of the lake with a rotenone preparation to eradicate the existing fish population. As the Dickinson Reservoir is the raw water supply for the city of Dickinson, approval was requested from the North Dakota State Department of Health, Bismarck, N.D., which asked for advice from USPHS.

Studies were undertaken at the Robert A. Taft Sanitary Engineering Center, Cincinnati, Ohio, to determine the feasibility of treating water that had been contaminated with a fish poison. The results of these studies have been published in the JOURNAL.[1, 2] The investigations showed that both toxicity and odor produced by the presence of fish poison formulations in raw water supplies can be reduced to safe and palatable levels by treatment with activated carbon in a water treatment plant. Data were also obtained which demonstrated that the duration of the problem at the plant would be relatively short, as both rotenone and solvent are largely dissipated within 1 week.

On the basis of the results of these studies, approval for the Dickinson Reservoir fish eradication project was granted to the State Game & Fish Department by the North Dakota State Department of Health. A request was made for USPHS to have an observer at Dickinson who could assist in evaluating the treatment of the water during the operation.

Fish Eradication Program

The fish killing operation was conducted by the North Dakota State Game & Fish Department. On Aug. 26, 1958, Dickinson Reservoir and influent streams were treated with a total of 5,500 gal of liquid concentrate fish poison.* Dickinson Reservoir, an impoundment of the Heart River, has a

* Pro-Noxfish, made by S. B. Penick & Co., New York, N.Y. The poison contains 2.5 per cent rotenone, 5 per cent cube and derris root extractives, 2.5 per cent sulfoxide, and 90 per cent inert materials.

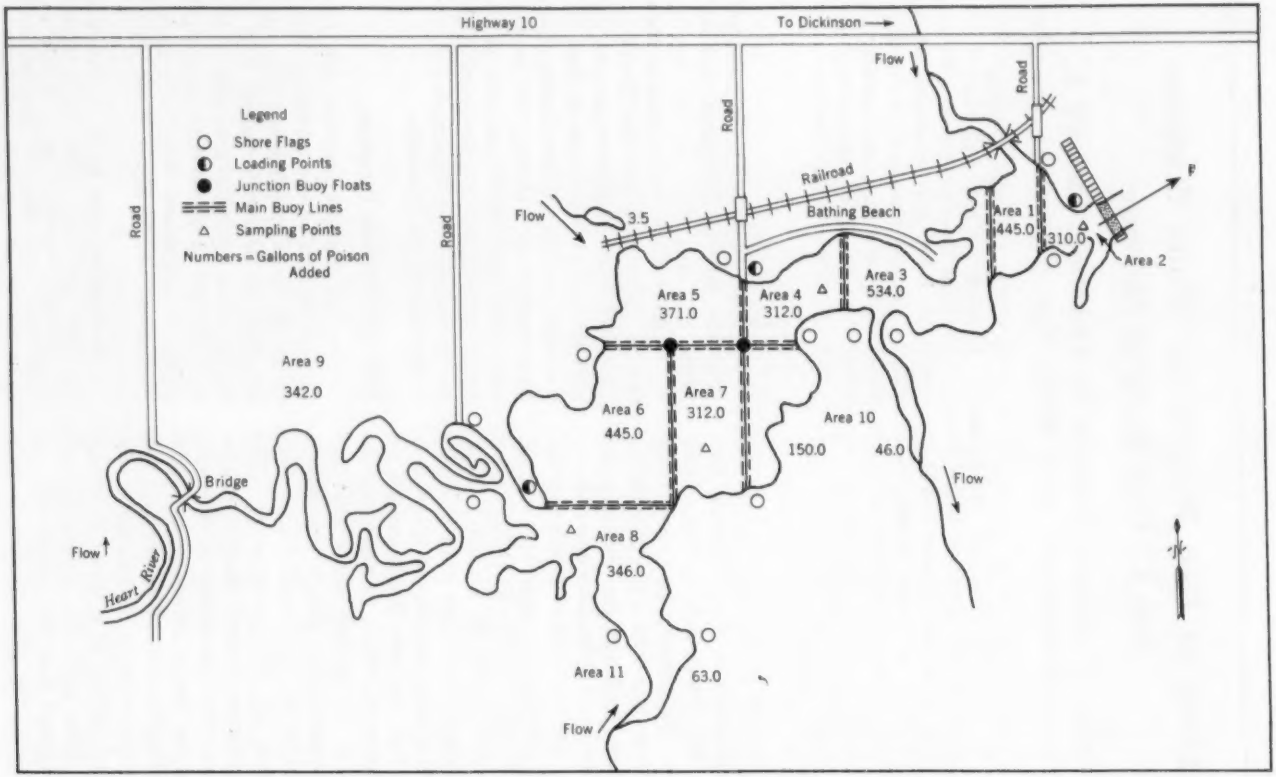

Fig. 1. Fish Eradication Program in Dickinson Reservoir

surface area of approximately 800 acres and a volume of approximately 5,800 acre-ft. To aid in the uniform distribution of the fish poison, the lake was sectioned into numbered areas (Fig. 1), and quantities of poison, calculated to produce a concentration of 2.0 mg/l, were apportioned to each area.

The chemical was sprayed from boats equipped with a motor-driven pump, a 55-gal drum containing the toxicant, and two discharge spray nozzles. Rapid operation of the boats repeatedly over the surface of the lake helped to emulsify and distribute the chemical. Shallow areas of the lake and tributary streams were treated by hand spraying.

Within a half-hour of the application of the poison, fish appeared on the surface. By mid-afternoon, when the toxicant had been applied to the main part of the lake, the crew started to harvest the floating fish. Poisoning of the tributary streams was continued for an additional day. The harvested fish consisted largely of carp, suckers, bullheads, and small bluegills. Also, a limited number of game fish were harvested. Practically all the visible dead fish were removed from the lake and trucked to a rendering plant. Fish continued to appear on the surface of the lake in large amounts for the next 2 days, and in lesser amounts for several days thereafter. It is estimated that 70 tons of dead fish were recovered. The lake was restocked with game fish in the spring of 1959.

Water Treatment Plant

The water supply for Dickinson, which has a population of approximately 8,000, is obtained from the Dickinson Reservoir. The water intake is approximately 15 ft under water near the concrete dam, and the water flows by gravity through a 16-in. conduit for a distance of 6,840 ft, until it enters the water treatment plant. Figure 2 is a schematic diagram of the water treatment plant. The treatment capacity of the plant is 2 mgd. In addition to the surface supply, wells are available which can produce 0.8 mgd. A storage capacity of approximately 1.3 mil gal is provided in the distribution system.

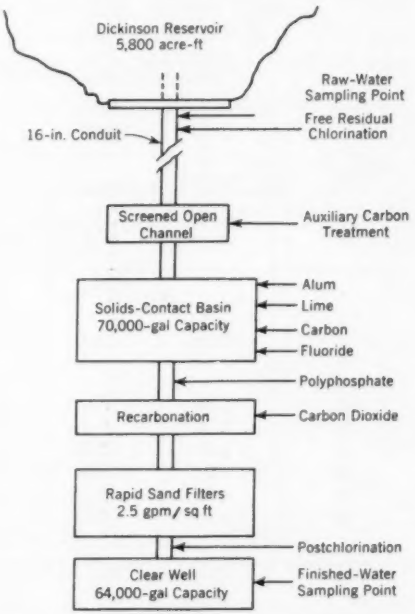

Fig. 2. Dickinson Water Treatment Plant

The recarbonation equipment was not operating during the study.

Facilities adequate for free residual chlorination are provided at the raw-water intake. Three dry-chemical feeders serve to add alum, lime, and activated carbon. A small pump adds liquid hydrofluorosilicic acid directly to the solids-contact basin. Water leav-

ing the basin is treated with a polyphosphate compound for corrosion control. Recarbonation facilities are provided but were not operating during the fish eradication program. The standard rapid sand filters were operated at a rate of 2.5 gpm/sq ft. Facilities for postchlorination are also available at the clear well.

Because the existing chemical feeder for activated carbon could feed a maximum of only 10 mg/l, an additional feeder that could supply the required 60 mg/l of carbon was improvised. The auxiliary feeder consisted of a 55-gal steel drum equipped with a mechanical stirrer and an outlet controlled by a valve located at the bottom of the drum. The feeder was located astride an open channel carrying raw chlorinated water to the solids-contact basin. The valve was adjusted to deliver at the rate of 50 gal of carbon slurry in 2 hr. The amount of carbon fed to the coagulation tank was controlled by varying the amounts of carbon added to the drum.

Laboratory Control

In addition to the normal control tests, two types of determinations were made for control of the water treatment: (1) quantitative analysis of the toxic component in both the raw and finished waters by fish bioassay and (2) determination of the threshold odor numbers of the raw and finished waters. Additional samples were also obtained from the lake at various points and depths, to assess the distri-

TABLE 1

Activated-Carbon Treatment of Fish Poison Emulsion *

Carbon Added (M) mg/l	Odor†			Fish toxicity‡		
	Removed (X)	Remaining (C)	$\frac{X}{M}$	Removed (X)	Remaining (C)	$\frac{X}{M}$
	Fish Poison Concentration—*mg/l*					
2				0.75	1.25	0.38
4				1.40	0.60	0.35
5	0.99	1.06	0.198			
6				1.62	0.38	0.27
8				1.81	0.19	0.23
10	1.74	0.31	0.174	1.85	0.15	0.19
15	1.74	0.31	0.116			
21	2.00	0.05	0.095			
30	2.01	0.04	0.067			
40	2.02	0.03	0.051			
75	>2.04	<0.01				

* The letter symbols refer to Freundlich's equation:

$$\frac{X}{M} = KC^{\frac{1}{n}}$$

in which X represents the units of adsorbate held by the carbon; M, the carbon weight by which adsorption was effected; and C, the concentration of substance remaining unadsorbed at equilibrium. The two parameters K and $\frac{1}{n}$ are particular to the system.

† The initial concentration of fish poison was 2.05 mg/l.
‡ The initial concentration of fish poison was 2.0 mg/l.

bution of toxicant, and from several points in the water distribution system. Three pairs of carbon filters were installed and operated to obtain an estimate of the extractable organic material in the raw and finished waters before, during, and after the fish eradication project.

Determination of Treatment Dosage

Before the fish kill operation had started, laboratory studies were made to determine the dosage of activated carbon which would be required to produce safe and palatable water. The methods used were described in previous articles.[1, 2] A fixed concentration of the fish poison formulation that would be used in the lake was added to the test water and was then treated with increasing amounts of activated carbon. The degree of removal of toxicant and odor was determined by fish bioassay and from threshold odor numbers, respectively. The data were used to plot carbon adsorption isotherms according to the Freundlich equation, which then made possible the computation of the carbon dosages that would be required to reduce any concentration of the fish poison to any desired level (Table 1). The resulting carbon adsorption isotherms are shown in Fig. 3. The parameters (Table 2) derived graphically from these curves were then used to compute the carbon dosages required to reduce any initial concentration of the poison to the desired residual. These curves are shown in Fig. 4. For fish toxicity, the permissible residual was one-half the 96-hr median tolerance limit (TL_m), as described in a previous article.[1] A threshold odor number of 1 was chosen as the desired residual odor. These two values are equivalent, respectively, to 0.03 mg/l and 0.013 mg/l of fish poison remaining in solution after carbon treatment. It is immediately apparent from Fig. 4 that treatment with activated carbon in sufficient amounts to remove odor would always result in the concurrent removal of fish toxicity, with more

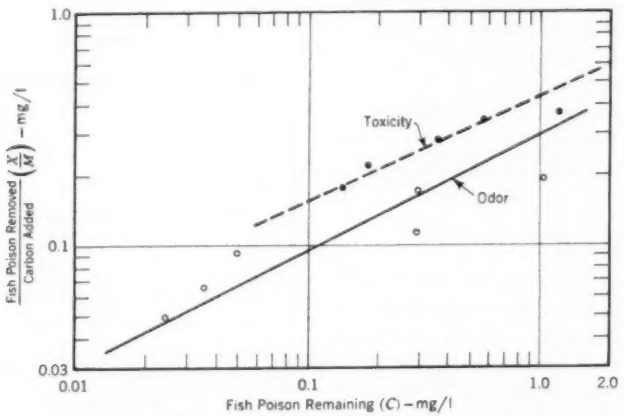

Fig. 3. Carbon Adsorption Isotherms for Fish Poison Used

See Table 2 for initial concentrations and parameters.

TABLE 2

Initial Concentrations and Parameters for Isotherms in Fig. 3

Determination	Initial Concentration of Poison *mg/l*	Parameters	
		K	$\frac{1}{n}$
Toxicity	2.0	0.41	0.43
Odor	2.05	0.30	0.50

than an adequate overdose of carbon. These laboratory data demonstrated that a minimum of 59 mg/l carbon would have to be added at the water treatment plant to reduce the odor of 2.0 mg/l of the fish poison to a residual threshold odor number of 1. Only 21 mg/l carbon would be needed to reduce the toxicity of 2.0 mg/l of the fish poison to one-half the 96-hr TL_m value.

Water Treatment

At 6:30 AM on Aug. 26, 1958, treatment of Dickinson Reservoir with fish poison was started. At the treatment plant, water was being supplied to the distribution system from wells operated at their capacity rate of 0.8 mgd and from storage facilities that had been filled the previous night to their capacity of 1.3 mil gal. In the meantime, water from the treatment plant was diverted to waste until a determination of the concentration of the fish poison in the raw water, both by fish bioassay and by odor value, could be made. The first three samples of raw water contained an average of 2.0 mg/l of fish poison as determined by bioassay, and had an average threshold odor number of 40. According to Fig. 4, 59 mg/l activated carbon would be needed to reduce this concentration to a safe and palatable level.

The treatment plant was tied into the system at 10:20 AM and was operated at one-third capacity. Activated carbon,* at the calculated concentration of 60 mg/l, was added with the use of both the regular and auxiliary feeding facilities. The water was coagulated with alum and lime. Free residual chlorination was maintained at the water intake, but as free chlorine is effectively removed by the large dosages of carbon, additional chlorine was added at the clear well. Samples of finished water, obtained at 11:00 AM and 11:50 AM, showed that the

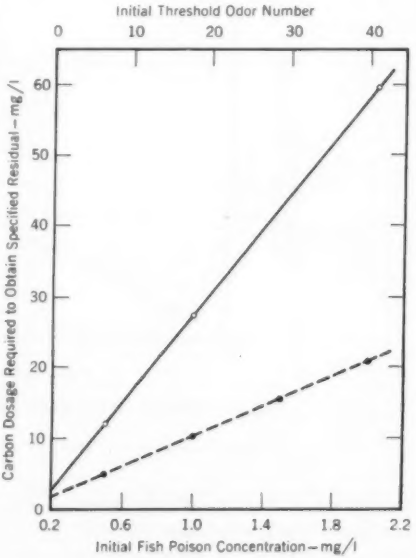

Fig. 4. Carbon Dosages for Removal of Fish Poison Used

The solid curve represents the residual threshold odor number of 1, equivalent to 0.013 mg/l of the fish poison. The dashed curve represents the residual of the poison equivalent to one-half the 96-hr TL_m, or 0.03 mg/l.

* Aqua Nuchar A, made by West Virginia Pulp & Paper Co., New York, N.Y.

raw water was being successfully treated with the predicted amounts of carbon. No fish toxicity could be found in either of these samples, and the threshold odor numbers were 1.5 and less than 1.0. Full operation of the plant was started at 4:00 PM and continued until 1:30 the next morning, when the plant was shut down for the night. Water treatment was resumed at 8:15 AM on August 27.

Operation of the plant was, unfortunately, not without mishap. A combination of two events made the first day less than completely successful. During the period 11:00 AM–1:30 PM, water entering the plant was unusually heavily contaminated with the fish poison. The concentration of poison rose sharply to more than 6.4 mg/l, and two samples showed a threshold odor number of 90 and 74, respectively. At about the same time, unnoticed difficulty with the auxiliary carbon feeder caused the carbon dosage to drop from 60 mg/l to approximately 17 mg/l. These events allowed inadequately treated water to enter the distribution system. Finished-water samples taken during the period 1:30 PM–5:00 PM showed a rise in threshold odor number to a peak of 7.0, and a decline again to less than 1 when proper carbon treatment was resumed. During this period, two consumer complaints were received, the only ones during the entire operation.

Fish Toxicity

A summary of the laboratory data is shown in Fig. 5. Except for the excessively high value of more than 6.4 mg/l obtained in the early hours during poisoning of the reservoir, the concentration of fish poison averaged 2.0 mg/l during the first day. Thereafter, the concentration of poison in the raw water decreased at a uniform rate, until at the end of 3 days, less than 0.1 mg/l, which was nontoxic to the test fish, remained. The percentage of poison remaining plotted against elapsed time is linear on semilogarithmic paper. From this curve, the half-life of the rotenone compounds was shown to be less than 1 day (0.75) under the conditions existing during the study. At no time during the entire study could any toxicity to fish be detected in the treated-water samples. A total of twelve samples was consistently nontoxic to the test fish during 24 hr of exposure.

TABLE 3

Distribution of Fish Poison in Dickinson Reservoir

Date—1958	Area*	Depth *ft*	Temperature °C	Fish Poison Concentration *mg/l*
Aug. 27	2	—†	19.9	0.82
	2	12	19.8	0.90
	2	25	19.7	0.94
	4	—†	19.8	0.85
	4	5	20.0	0.84
	4	10	20.4	0.84
Aug. 28	2	—†	19.8	0.65
	2	12	20.0	0.70
	2	24	20.1	0.78
	4	—†	20.0	0.53
	4	5	20.1	0.43
	4	10	20.0	0.57
	7	—†	20.0	0.65
	8	—†	20.0	0.57

* See Fig. 1.
† Surface sample.

Samples for the determination of fish poison concentration were obtained from various areas (Fig. 1) and at various depths in the reservoir. Unfortunately, lack of time did not permit sampling during the first day. The data (Table 3) show that by the second day excellent distribution of poison

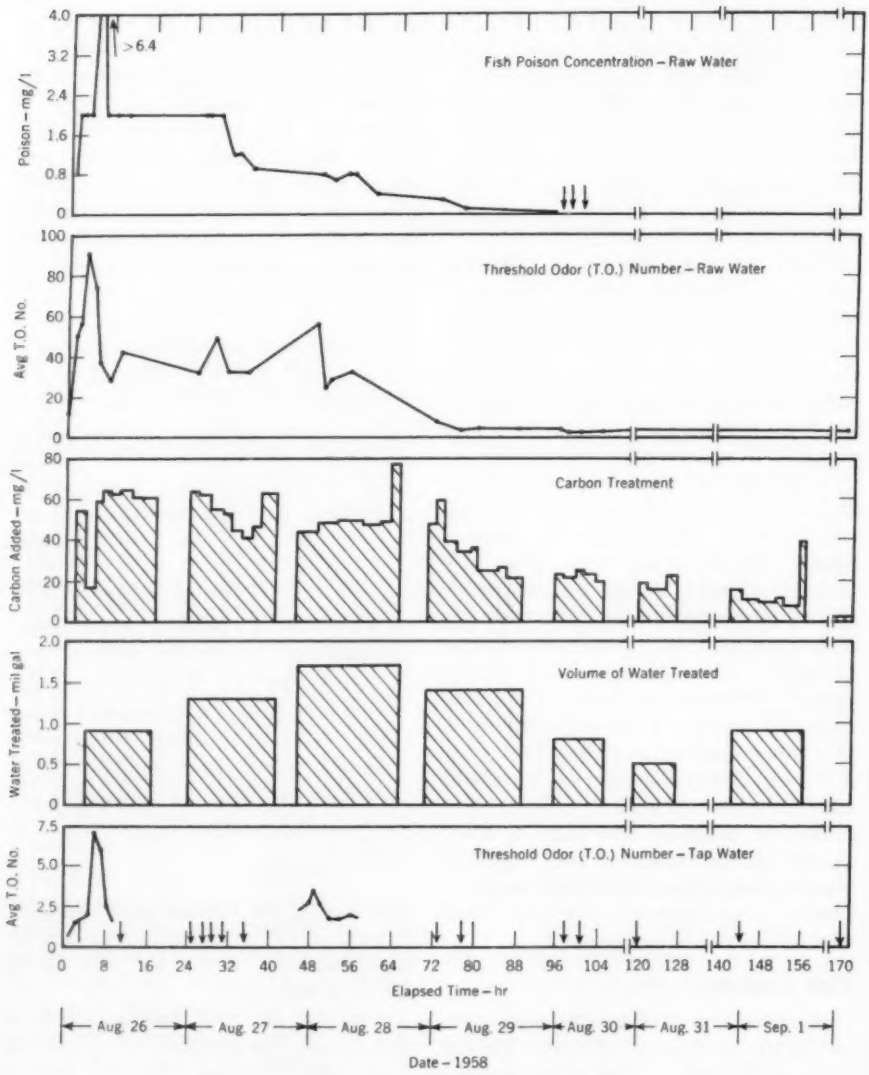

Fig. 5. Summary of Laboratory Data at Dickinson Plant

Arrows in the top illustration indicate values less than 0.1 mg/l. Arrows in the bottom illustration indicate threshold odor numbers less than 1.

had been achieved in the areas tested in the lake.

Odor of Raw Water

Uneven application of fish poison near the raw-water intake produced a slug of heavily contaminated water with threshold odor numbers ranging up to 90. If two high values are omitted, the average threshold odor number of the raw water on the first day was 42. At the end of 3 days, odor had decreased to a threshold odor number of 5, although the odor typical of the poison was still discernible. The threshold odor number had decreased to 4 at the end of 6 days; no odor of solvent could be detected, but a marshy odor, characteristic of the raw water before fish poison had been added, was observed.

Activated-Carbon Dosages

After determination of the poison content of the raw water by fish bioassay and by threshold odor number, the amount of carbon required could be predicted by reference to the carbon dosages shown in Fig. 4. These data are shown in Table 4. Good control of the applied carbon dosages as compared to predicted dosages was maintained during the first 3 days. Excessive amounts of carbon were applied on the following days because of a combination of events. During this period, the threshold odor number was dropping very rapidly, and water usage was being curtailed because of the onset of cooler weather. The lag in time introduced by the collection and analysis of the samples and the fact that the improvised activated-carbon feeder could not be accurately controlled also account for the high carbon dosages. At the end of 7 days, it was possible to reduce the carbon dosages to the normal amounts of 3–4 mg/l used before the fish eradication operation.

TABLE 4

Predicted and Applied Activated-Carbon Dosages

Date—1958	Avg T.O.* No.	Avg Fish Poison Concn. *mg/l*	Carbon Concn. Predicted *mg/l*	Avg Carbon Concn. Applied *mg/l*
Aug. 26	42	2.0	61	61
Aug. 27	35	1.5	52	53
Aug. 28	35	0.7	52	48
Aug. 29	5	0.18	10	36
Aug. 30	3	<0.1	8	23
Aug. 31	4		9	18
Sep. 1	4		9	12

* Threshold odor.

TABLE 5

Threshold Odor Numbers of Samples From Distribution System

Date—1958	Distance from Plant *ft*	Avg Threshold Odor No.
Aug. 28	3,500	2
	3,800	2
	7,400	2
	8,100	<1
Aug. 29	3,500	<1
	3,800	<1
	7,400	<1
	8,100	<1

Odor of Finished Water

When sufficient activated carbon, as predicted from raw-water analyses, was actually applied, water with a threshold odor number of 1 or less was invariably produced. On two occasions, inadequate carbon treatment immediately resulted in finished water with threshold odor numbers greater than 1.

The causes of the high threshold odor values during the first day have

TABLE 6

Chemical Characteristics of Waters

Date—1958	Hardness (as $CaCO_3$)—*mg/l*		Alkalinity (as $CaCO_3$)—*mg/l*		pH		Turbidity		Free Chlorine in Tap Water *mg/l*	Fluoride in Tap Water *mg/l*	Temperature in Raw Water °F
	Raw Water	Tap Water	Raw Water	Tap Water	Raw Water	Tap Water	Raw Water†	Tap Water			
Aug. 24*	98	83	158	134	8.1	8.9	25	0	0.5	1.2	72
Aug. 25*	98	100	156	156	8.1	8.9	25	0	0.5	1.2	72
Aug. 26	98	122	168	156	8.1	8.9	80	0	1.0		71
Aug. 27	99	103	157	157	8.1	8.9	30	0	0.75	1.2	72
Aug. 28	99	96	157	138	8.1	8.9	25	0	0.75	1.2	72
Aug. 29	99	103	157	154	8.1	8.9	25	0	0.5	1.2	72
Aug. 30	98	92	170	140	8.5	8.9	22	0	1.0	1.2	65
Aug. 31	98	118	180	158	8.3	8.9	10	0	1.0	1.2	70
Sep. 1	97	104	168	155	8.1	8.9	12	0	0.9	1.2	69

* Before addition of fish poison.
† Turbidity showed a steady decrease to a minimum of 1 on September 23.

already been mentioned. On August 28, 2 days after the application of the fish poison, a rise in threshold odor number of the finished water to a maximum of 3.5 was observed. This lapse in adequate treatment occurred because a constant amount of carbon was being added during a period when water pumpage was at its maximum during the time of the study. An average of 48 mg/l carbon was applied when the predicted requirement was 52 mg/l, as shown in Table 4. No consumer complaints occurred during this period, because dilution in the distribution system served to reduce the threshold odor number at the household water tap. A limited number of samples were obtained from various points in the distribution system (Table 5).

Plant Data

Data shown in Tables 6 and 7 summarize the chemical characteristics of raw and tap waters both before and during the fish eradication operation. The listed chemical and physical quali-

TABLE 7

Chemicals Added at Treatment Plant

Date—1958	Alum *mg/l*	Lime *mg/l*	Fluoride *mg/l*	Polyphosphate *mg/l*	Activated Carbon *mg/l*	Chlorine *mg/l*
Aug. 24*	69	88	1.1	2.0	3.4	7.3
Aug. 25*	60	80	1.4	2.0	7.0	8.9
Aug. 26	68	81	0.85	2.2	61	10.0
Aug. 27	73	73	0.88	2.2	53	7.8
Aug. 28	73	80	1.1	1.5	48	9.5
Aug. 29	60	77	1.0	1.9	36	8.0
Aug. 30	45	76	0.95	2.1	23	9.1
Aug. 31	71	71	1.2	2.1	18	7.1
Sep. 1	65	52	1.1	1.8	12	8.3

* Before addition of fish poison.

ties of the waters were not affected, except for raw-water turbidity (Table 6) which showed a steady decrease to a minimum of 1 on Sep. 23, 1958. Eradication of fish that stir the bottom mud probably accounts for the clarification of the lake.

Considerable amounts of carbon were carried over from the flocculator to the sand filters, but without any detriment to the filter runs. Before addition of fish toxicant, filter runs averaged approximately 42 hr; the duration of runs during the project was 27–66 hr, with an average of 46 hr.

During the period Aug. 26–Sep. 1, 1958, when activated-carbon dosages greater than normal were required, a total of 2,400 lb was added. At 16 cents per pound, the cost of activated carbon for removal of toxicity and odor of 2.0 mg/l of fish poison amounted to $384. As at this time of the year the normal use of carbon averaged approximately 3 mg/l, the net cost of chemicals to the treatment plant attributable to the fish eradication operation came to $354.

Carbon Filters

Carbon filters,[3] installed on the raw-water line at the dam site just prior to the point of chlorination and on the finished-water line at the treatment plant, were operated before, during, and after the fish eradication operation. The concentration of chloroform-extractable organic material from these filters is shown in Table 8.

Concentrations of chloroform-extractable organic material in both the raw and finished waters of the first pair of samples are what might be expected from an unpolluted surface water in which materials of natural origin are invariably present. A quantity of light, oily material suggestive of lubricating oil produced the apparent anomaly of more organic material in the finished-water sample than in the raw-water sample. The almost tenfold increase of organic materials in the raw water obtained during, and for some time after, the fish eradication operation confirms the application of the fish poison. More revealing, however, is the low value for the finished water during this period, ample evidence of the effectiveness with which the water plant removed the contaminating fish poison. The increased organic content of the raw water obtained a month after the fish eradication operation is probably the result of the decomposition of poisoned fish that had not been harvested. The quality of the finished water, as reflected by its organic content, remained unchanged from the period prior to the fish kill.

TABLE 8

Concentration of Chloroform-Extractable Organic Material in Raw and Finished Waters

Date of Filter Operation—1958	Concn. of Chloroform-Extractable Material μg/l	
	Raw Water	Finished Water
Aug. 13–23	46	75
Aug. 26–Sep. 26	435	38
Oct. 1–Nov. 5	151	75

Further evidence of the efficiency of the water plant in removing the fish poison is provided by infrared spectrograms that were obtained on all of the chloroform extracts in raw and finished waters and on the fish poison itself. Selected spectrograms are shown in Fig. 6. Spectrogram 1,

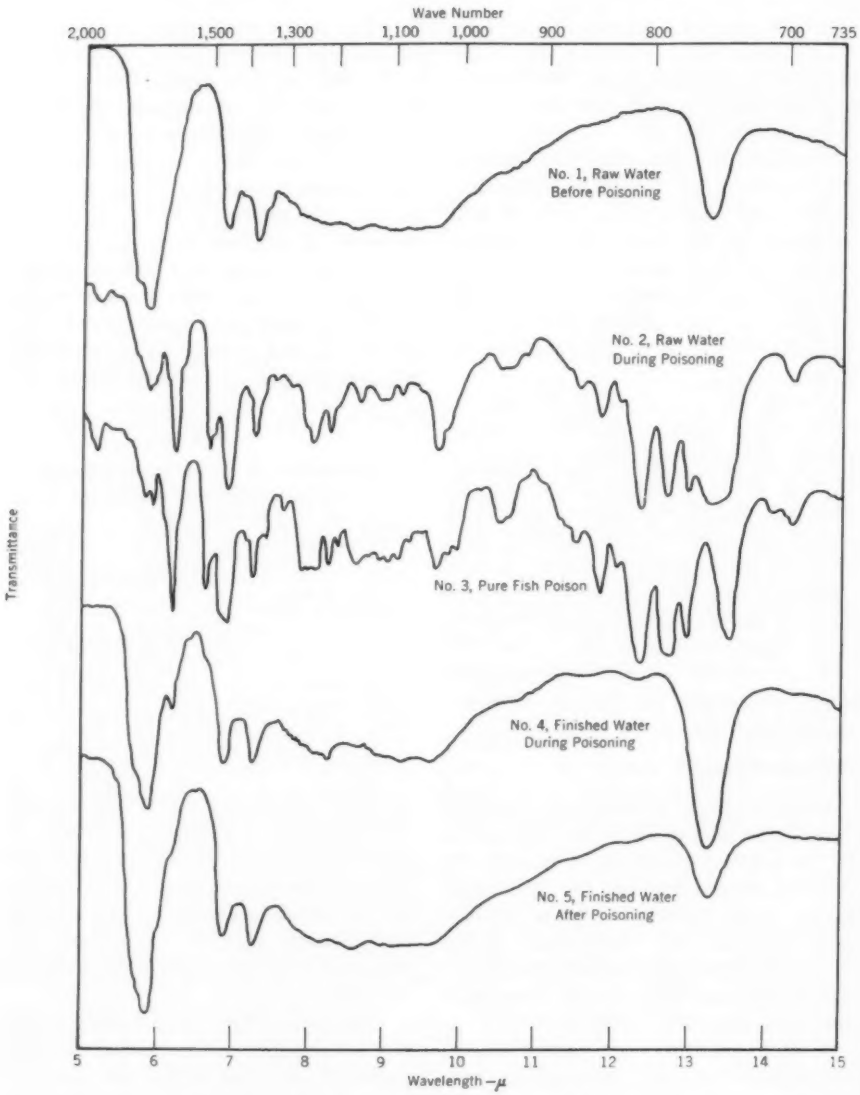

Fig. 6. Infrared Spectra of Carbon Filter Extractions

The striking similarity between Spectrograms 2 and 3 is evidence that fish poison was present in the raw-water extract.

prepared from the chloroform extract of the raw water prior to the fish poison application, shows no fine structure and is typical of the kind of spectrogram obtained from a complex mixture of organic material normally present in an unpolluted surface water. But Spectrogram 2, from the raw water during the fish eradication operation, exhibits many well defined peaks. Indeed, the striking similarity of this spectrogram with that obtained for pure fish poison, Spectrogram 3, is convincing evidence of the presence of this fish poison in the raw-water extract. Spectrogram 4, of the extract from finished water obtained during the fish eradication operation, shows no evidence of the presence of fish poison. Efficient removal at the water plant had reduced the residues to undetectable amounts. Spectrogram 5, obtained for the extract of raw water more than a month after the fish eradication operation, shows no detectable change from Spectrogram 1.

Summary and Conclusion

During the period Aug. 26–Sep. 1, 1958, the water treatment plant at Dickinson, N.D., successfully treated water that had been contaminated with an average of 2.0 mg/l of a commercial fish poison formulation. A total of 5,500 gal of the fish poison was added to Dickinson Reservoir—volume approximately 5,800 acre-ft—as the first step in the fish management program. Treatment at the water plant consisted of adding 61 mg/l activated carbon, which effectively reduced the toxic and odorous components to safe and palatable levels. This dosage was predicted as adequate after laboratory study. The components toxic to fish decomposed at a rapid and uniform rate and could not be detected in the raw water at the end of 3 days. The odor component of the fish poison dissipated more slowly, but it also could not be detected at the end of 6 days. During this time, decreasing dosages of carbon were required for proper treatment of the water. Except for two instances when raw water was known to have been treated with inadequate amounts of carbon, the finished water had a threshold odor number of less than 1, and was uniformly nontoxic to fish. No operational difficulties were encountered at the plant, and operation of the plant was back to normal within 8 days. During this period, the cost of activated carbon required, in excess of that normally used, was $354.

A carbon filter study of the entire operation showed that the water plant had effectively removed the components of the fish poison from the raw water. Corroborative evidence was obtained by infrared confirmation of both the presence of the fish poison in the raw water and its absence in the finished water. One month after the fish eradication operation, no change could be detected in the quality of the raw water.

Consumer acceptance of the water was good. Except for two complaints received during the first day after an operational failure, no objections were received at the plant. Public acceptance was facilitated by news coverage in the local newspaper that had prepared the public for a possible slight change in taste of the water. Random questioning of townspeople revealed that although most consumers detected no odors in the water, a few individuals were aware of the "kerosenelike" flavor. This reaction was not unexpected, because a residual threshold odor number of 1 indicates that some odor is detectable.

This study substantially confirmed the predictions of laboratory studies [1, 2]—namely, that with adequate activated-carbon treatment a water plant is fully capable of producing safe and palatable water from a supply that has been contaminated with a fish poison formulation.

Acknowledgments

The authors wish to express their appreciation to Robert Francis, chemist at the North Dakota State Department of Health, who helped with the laboratory control during the study; to Dale Henegar, chief of the Fisheries Unit of the North Dakota State Game & Fish Department, and his staff, who supplied the native test fish used in the bioassay tests and who aided in many other ways; to F. M. Middleton, in charge of Organic Contaminants Unit at the Taft Center, and staff, for the carbon filter work; and to Peter Pfau, superintendent of the Dickinson water treatment plant, and his staff, who performed an outstanding job of operating the water plant under adverse conditions and who willingly contributed their efforts in making the project a successful one.

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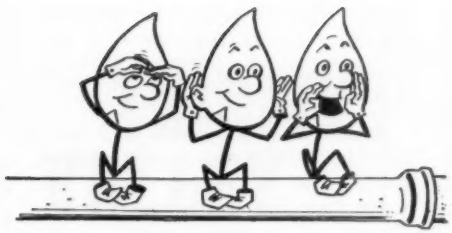

Percolation and Runoff

'The Effluent Society' is not the newest name of WPCF, nor indeed of AWWA, but a characterization of our present civilization employed by Ed Cleary, executive director and chief engineer of the Ohio River Valley Water Sanitation Commission, at the National Water Pollution Conference last December. "The Affluent Society" it had better continue, too, if the multibillion-dollar solutions to present pollution problems discussed at the conference are ever to be accomplished. "The Fluent Society" it, meanwhile, seems to be becoming, both in the amount of water required and in the number of water words employed.

Actually, the conference, which attracted a reported 1,500 water and health experts, was most productive in giving a further boost to public interest in water supply, its conservation and augmentation. Aroused by the well publicized activities of the Senate Select Committee on National Water Resources and the Office of Saline Water and ministered to by the growing program of the National Water Institute, organ of the WSWMA public relations committee, public interest in water supply is now at an all-time high. And though too much of that interest may have been aroused by misinformation concerning the "imminent shortage of water" it behooves us to take advantage of the fact that attention is the first requisite of education and demonstrate that not water itself, but the facilities to make it available where it is needed when it is needed and its protection against pollution constitute the problem.

Hail "The Fluent Society."

Fluency is virtually epidemic these days in the field of desalinization. Most illustrious of those infected is President-elect Kennedy who has seized upon the oceans to wash out the memory of space. "Can you imagine," he said, "what that (the breakthrough in sea-water conversion) will do for the world? The first country that's able to do that will earn far more prestige than we lost being second in outer space." Meanwhile, Vice-President-elect Johnson, who introduced legislation last spring to increase research funds in the field, displayed the same symptoms in an article in the New York *Times* magazine section, noting "There is no newer or more vital frontier for any of us than the one we must cross to a lasting abundance of fresh water for all mankind." And, as if to prove that water supply is at

(*Continued on page 36 P&R*)

(Continued from page 35 P&R)

last taking its proper place next to motherhood among political issues, Vice-President Nixon sounded an urgent call to accomplish the same goals, while, in Congress, Republicans and Democrats argued heatedly on which was first to call for this important quest.

More power to them, we say, but before they embark on the $100-billion program that Dr. A. L. Miller, director of the Office of Saline Water, suggests as a starter to meet the country's demand for water in the next two decades, we hope they will take a look at some of the suggestions Abel Wolman makes in the lead article of this issue. When we think of how much the nation's water utilities could accomplish in solving their problems with Dr. Miller's $100 billion, we're almost ready to support *Barron's* financial weekly in its editorial suggestion that the federal desalting program be dropped and the problem left to private enterprise, which, *Barron's* says, is now being discouraged by the government competition in the field. On the other hand, past experience indicates that, if the $100 billion were to be left where it will have to come from, water utilities would see very little of it.

Meanwhile, until fluency has developed into influence, too many water utilities will manifest financial immunity.

Rhabdomancy, the art of water witching or dowsing or divining, is apparently in a sad state of decline—or perhaps it has gone underground. At any rate, what once was a rich source of interesting P&R grist has almost completely dried up. Thus, in the past two years only two real "gone" water witchers have been featured in the news—and one of these, as will be suggested by the following "complete" news story:

> NEWTON, Miss. (UPI)—Robert McDaniel, 56, still uses a divining rod to locate water in Newton County.
>
> He maintains he has never missed in 20 years of "water witching."

hardly provides the kind of colorful story that once turned up at least twice a month. Even the other—the story of a Mrs. Adrian Wormley of Ransom, Ill., who uses two pieces of copper wire instead of a wooden wand and started witching as a hobby at the suggestion of her husband and later became a professional and appeared on Radio WLS, Chicago, homemaker's hour to demonstrate her prowess and . . . oh well (sometimes anyway)—isn't exactly a normal witch either. In the first place, she has the temerity to suggest that up to now there have been only eight accurate water dowsers in the United States. In the second place, wishing to become the ninth, she admits that she has failed a third of the time she has tried and that her estimates of depth can be as much as 10 per cent off. Of course, she does absolutely claim that all her successful wells have produced from 4 to 300 gpm.

There is one other dowsing tale we haven't quite digested or classified as yet. That is one which appeared in the *exPUDiter,* publication of the Skagit County Public Utility District, Mount Vernon, Wash., reporting the verification by Manager Fred Ovenell of a dowse by Canadian diviner Jimmie Bell in the Deception Pass area of Washington. We have, however, no report on whether water was found.

Most of the divining these days, though, is too scientific to be interest-

(Continued on page 38 P&R)

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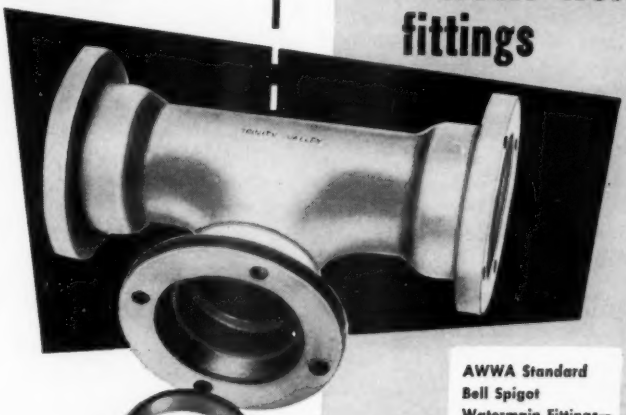
for all cast iron water works fittings



AWWA Standard Bell Spigot Watermain Fittings— 2" through 36".

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TRINITY VALLEY IRON AND STEEL COMPANY

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(Continued from page 36 P&R)

ing. From South Bend, Ind., for instance, comes the story of a Notre Dame geologist and his aviatrix wife who use aerial color photography to detect ground water and are perfecting their system under a $6.500 grant from the Office of Naval Research. And from the New Mexico Institute of Mining and Technology we learn of an electronic water witch perfected by Dr. Paul Kintzinger and a group of hydrologists to send electronic fingers hundreds of feet below the surface to search for water and determine not only its exact depth, but whether or not it contains undesirable chemicals. Called the induced electrical polarization technique, this is hardly romantic, much less rhabdomantic.

Give us, we were going to say, the good old days, but, remembering the past history of divining, we'll await the good new ones, when nylon, PVC, uranium, and radioisotopes become part of the "cool" dowser's kit.

L. S. Morgan has accepted a position as director of environmental health for the Bucks County (Pa.) Department of Health. Formerly he was chief of the mine drainage section for the State Department of Health. A former AWWA director (1953–56) and recipient of the Fuller Award (1949), he is currently secretary-treasurer of the Pennsylvania Section.

Realists these days recognize that no ordinary homeowner, however handy, can any longer be expected to change a worn washer on a modern faucet. Not only would the tools required to perform the operation probably cost more than the sink itself, but the structure of the valve mechanism has been so refined that even the ethical professional plumber will replace the entire unit rather than build up the time to substitute a new penny washer. That way, stopping a drip will usually cost less than $10.

In obvious recognition of this situation and of the fact that one may have to wait until one's plumber returns from a Florida vacation to get the job done, the New York *Times'* homemakers' page recently included the following intelligence:

> A leaky faucet can wear an unsightly groove into a porcelain sink in a year's time, according to the Porce-Cote Corp. of Uniondale, L.I. Once the porcelain is worn away, the rusting process on the underlying metal is accelerated.
>
> If a leaky faucet cannot be repaired immediately, the company advises placing a piece of rubber or plastic on the spot where the water hits. If the surface is damaged, Porce-Cote offers a home repair service.

We really prefer the string technique, whereby one can lead the drop from the faucet direct to the drain by running a string or thread from one to the other. But as a water works realist we wonder if a meter that registers drips isn't worth while after all.

Three notable omissions from AWWA's new Animal Identification Manual * have already been called to our attention:

First, from St. Louis, Mo., has come word of a motile, macroscopic, multicellular animal recently isolated from the water of the treatment plant settling basin with the aid of a winch. On examination, the 150-lb specimen was identified as an eight-point buck by a

* Based on the Ingram-Bartsch article (December 1960 issue), to be published next month as AWWA M7, price $1.00.

(Continued on page 40 P&R)

MODEL HD—SOUTHERN PATTERN
⅝", ¾ x ¾", ¾", 1", 1½", 1½", 2" & 3" sizes.

MODEL HF—NORTHERN PATTERN
⅝", ¾ x ¾", ¾" & 1" sizes.

Always Specify a **Conical Disc Piston*** *to be Sure of Maximum Revenue*

Shown here are Model HD 1½" size (left) and 2" size (right).

Hersey Disc Meters are renowned for their 1-piece, metal reinforced, perfectly balanced, conical shape disc piston which does not require a mechanical guide. These vulcanized hard rubber "arch-type" design pistons allow lighter construction and assure increased sensitivity, greater accuracy and longer life.

Hersey Disc Type Meters with conical shaped pistons provide direct, unrestricted ports and waterways. The operating parts are well balanced, so that frictional losses are reduced to a minimum and internal errors, at varying rates of flow, are practically eliminated. Result: Large capacity, extreme sensitiveness, consistent accuracy, low maintenance, long life. Write for new bulletin giving complete engineering information.

*Conical Disc Piston for 2" size Hersey Meter shown in picture above.

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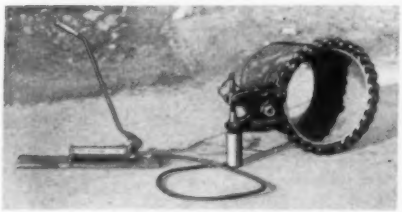

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No rotating—all you do is wrap the chain around the pipe, engage it in the cutter's upper jaws, adjust out slack and operate pump.

JUST
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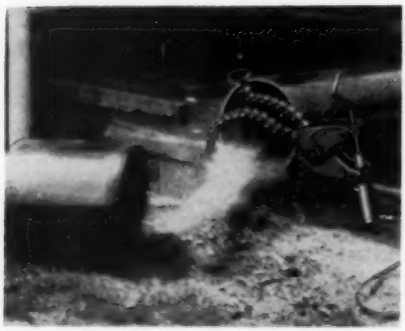
For pipe sizes up to 12" we recommend our Heavy-Duty Hydraulic shown here in the act of cutting a section of large diameter pipe.



The Wheeler Mfg. Corp.
P. O. Box 688
Ashtabula, Ohio

(*Continued from page 38 P&R*)

local representative of the humane society, who cancelled all plans for a venison roast by releasing the roastee forthwith.

Then, from Hayward, Calif., came a description of a new technique, employing an injection of tranquilizer, in the removal from a well water supply of a columnar-shaped five-appendaged animal. Upon examination in its sling, at the end of a heavy construction crane, the fifth appendage proved to be the tail of a pinto mare that had entered the supply after the well cover had been removed on Halloween.

Finally at Sulitjelma, Norway, the report was one of a whole horde of animals, found dead in the local reservoir, poisoning the water so that residents had to boil it for several days. These proved to be small rodents, 4 or 5 in. long, with short tails, furry feet, and short ears. Called "lemmings" they travel across the countryside of northern Scandinavia in large numbers.

Seen any since New Year's Eve?

Perseverance, a quality we have long credited to water works men in great abundance, seems to be equally characteristic of another occupational group—automobile racing drivers, that is. It took only two reports to convince us:

Monza, Italy, Oct. 1 (UPI)—A Fiat-Abarth-Farina roadster was swept off the Monza track by a thunderstorm today, but the driver, Umberto Maglioli, survived the crash and set an international and world record. The car, weighing only 1,170 lb, was engulfed by the storm as it sped around the overbanked track at 125 mph. The car turned to the right in midair and somersaulted twice before crashing. Maglioli pushed the car 2 mi

(*Continued on page 42 P&R*)

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(Continued from page 40 P&R)

to the finish line and set a 72-hr record for total distance—15,441.498 km.

Riverside, Calif., Oct. 16 (UPI)—Billy Krause drove his white birdcage Maserati to an easy triumph today in the $26,000 sports-car Grand Prix. The 27-year-old Long Beach machinist averaged 91.5 mph in the 2.9-liter machine and won $8,500. Time for the 203.05-mi race was 2 hr, 13 min, 10 sec, breaking the course record, set last year, by 3 min, 35 sec. Krause ran out of gas near the end of the victory lap and had to push the racer across the finish line, to the cheers of 80,000 lining the nine-turn, 3.2-mi course.

They could probably even push through adequate rates.

Qualifying examinations for diplomate candidates of the American Academy of Sanitary Engineers will be held on May 18, 1961, by the American Sanitary Engineering Intersociety Board, of which AWWA is a cosponsor.

The examination will consist of three parts, one dealing with basic engineering sciences, the second dealing with the area of specialization selected by the applicant, and the third an oral examination. Areas of specialization include public health and water supply and waste disposal.

Applications must be received by the board before Apr. 1, 1961. Forms may be obtained by writing American Sanitary Engineering Intersociety Board, Inc., 33 W. 39th St., New York 18, N.Y.

William N. Marshall Jr., formerly a senior engineer with the firm of Burns & McDonnell, has been named chief engineer and superintendent of the Kansas City (Mo.) water department. His predecessor was James B. Ramsey, who last fall joined the AWWA staff as an assistant secretary.

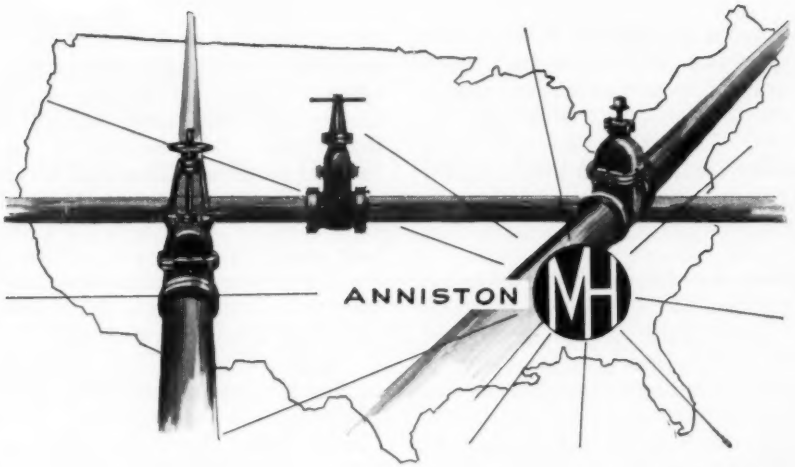
Stiffness in the joint showed up recently in two communities where joint administration of water and sewer accounts is in force:

At Oregon City, Ore., Samuel F. Newton, a resident who objected to paying for sewers he did not yet have, decided to pay only the water charges on his joint bill. Two months later he received a registered letter from the city returning the two $1 bills he had left to pay his water bill. With them was another bill carrying a charge of $2 for water and 50 cents for sewage. Thus, from the time he stopped paying the sewer charge because he had no sewers, Sam said, he had received 2 months of water service for which the city would accept no money, had the benefit of 2 months' service from the non-existent sewers, and had 68 cents' worth of cancelled stamps. He also pointed out that, if his neighbors followed the same procedure, they would have recovered a major portion of the $24,000 they had paid in for sewers they also didn't have. Asked what he expected the city to do to force him to pay, Sam suggested that they might shut off his sewer service.

At Marinette, Mich., meanwhile, when the city council decided to add a 70 per cent sewer service tax to the water bill in order to balance the budget for 1961, the four-member Marinette Water Utility Commission resigned. Indicating that they did not question the council's authority to levy the tax, even without consulting them, the commissioners noted that they had resigned because they opposed the tax and were unwilling, therefore, to undergo the certain criticism of a public that would not understand or agree with it.

Some people expect to get married without getting a mother-in-law.

(Continued on page 44 P&R)



a better Mouse Trap

The original use of iron body, bronze mounted valves in America came about through the discovery of steam power and the resulting industrial expansion about 1850 and later. In fact, the valves of those times were called "steam specialties."

Then in 1920, one of the old line valve companies was liquidated and much of the equipment for manufacturing iron body, bronze mounted valves and fittings was sold to a group of men in Anniston, Alabama.

That was the beginning of M & H! The first M & H valves were made in 1926, but difficulties were encountered and by 1932 annual sales volume was only $95,000. During that year of business depression, the Company was reorganized, refinanced, its products redesigned, its foundry and plant subsequently mechanized – and M & H started to grow!

It is the old story of "build a better mouse trap and the world beats a path to your door" – except in this case it was a better valve and hydrant. Today, 28 years later, M & H dollar volume of sales amounts to many millions annually.

Such growth in sales volume is no accident. Hard-boiled engineers and utility management buy valves where they get most for their dollars.

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SUBSIDIARY OF WALWORTH

(Continued from page 42 P&R)

Victor A. Appleyard, formerly chief of operations at Philadelphia's water department, is now executive manager of the Chester (Pa.) Municipal Authority. During his seven years at Philadelphia he was largely responsible for the development of automation in the water system.

Reservoir weedy? Put aside that weedicide! Get a manatee! Bothered by water hyacinth, Johnson grass, other plant pests? Put your manatee out to graze. In no time at all you'll have a bare bottom.

The manatee, lest you've forgotten or didn't even know, is an aquatic herbivorous mammal found in the rivers of Florida, Mexico, Central America, and the West Indies and hunted there for its flesh and oils. Its rear half resembles the tail part of a whale; its front half has an odd seal-like head, practically no neck, and two front limbs. At a distance it apparently is more attractive, for it is the creature that inspired the sailors' yarns of mermaids. Those who got a closer look named it a sea cow. Normally about 8 ft long, the manatee grows to lengths of 13 ft; it is a docile animal which has been kept as a pet.

Recently William H. L. Allsopp, a zoologist at the government fisheries laboratory in Georgetown, British Guiana, found that the manatee has a voracious appetite for weeds, eating 100 lb or more per day when it has the chance. To test the usefulness of this information, he put some manatees

(Continued on page 46 P&R)

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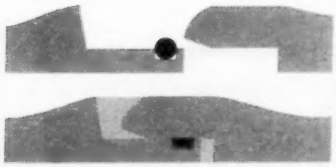

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A MEMBER OF THE CONCRETE PRESSURE PIPE ASSOCIATION

American PIPE AND CONSTRUCTION CO.

(*Continued from page 44 P&R*)

to work in the local drinking water reservoirs and found that they cleaned out the weeds in short order. In another test, two 7½-ft manatees were able to eat out a weed-choked canal, 22 ft wide by 4,800 ft long, in just 17 weeks, even breaking through a wooden barrier to get at their favorite food. No longer experimenting now, Allsopp has 46 manatees working for him and an order for 50 more placed with local fishermen.

Inasmuch as manatees are reported to be "slow and unenthusiastic in reproduction," we suggest that you waste no time in getting yours. And if you don't have a reservoir, or a big enough reservoir, we'll be happy to advertise your available manatee time.

William S. Peterson has retired as general manager and chief engineer of the Los Angeles Dept. of Water & Power, following 38 years of service with the organization. He had been general manager since 1955.

Laurie M. Leedom has been appointed engineer in charge of the Newark (N.J.) Division of Water Supply. He had been assistant division engineer. He was a recipient of AWWA's Goodell Prize in 1949.

Rolf Eliassen, professor of sanitary engineering, has been made acting head of the department of civil and sanitary engineering at Massachusetts Institute of Technology. He was head of the department's sanitary engineering division.

(*Continued on page 48 P&R*)

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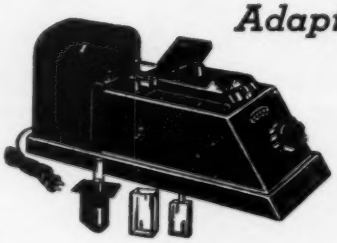
ELECTRIC PHOTOMETER



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(*Continued from page 46 P&R*)

Carl H. Chatters, retired city controller for Chicago (1955–60), died on Dec. 13, 1960, in New York City, where he had undergone surgery after being stricken three weeks earlier while attending a meeting of the American Municipal League. He was 62 years old. He had been executive director of the Municipal Finance Officers Association (1932–45) and also executive director of the American Municipal Association (1948–54).

Earl Devendorf, former director of the Bureau of Environmental Sanitation, New York State Health Department, died on Dec. 28, 1960, in Schenectady, N.Y., at the age of 72. He had served with the health department from 1918 to 1958. A member of AWWA since 1919, he was a Life Member, as well as a recipient of the Diven Medal (1943) and the Fuller Award (1943).

Employment Information

Classified ads will be accepted only for "Positions Available" or "Position Wanted." Rate: $1.50 per line (minimum $5.00), payable before publication. Deadline for ad copy: first of month prior to month of publication desired. To place ad, obtain "Classified Ad Authorization Form" from: Classified Ad Dept., Journal American Water Works Assn., 2 Park Ave., New York 16, N.Y.

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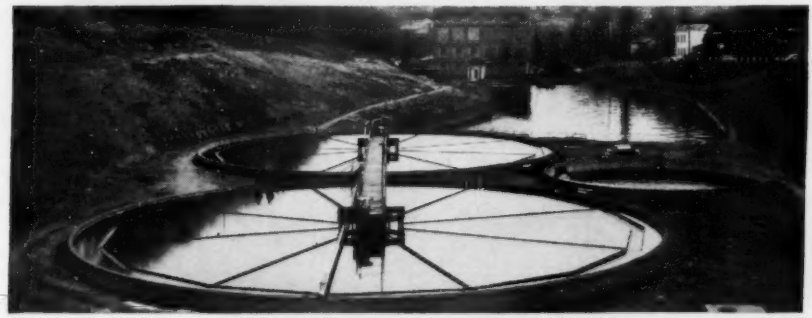

Here's how South Pittsburgh Water Company overcomes the tremendous problems posed by making Monongahela River water fit for domestic use. The water contains industrial wastes, sewage pollution and acid mine drainage—hardness ranges from 50 to 400 ppm and pH fluctuates continuously.

Serving more than 350,000 people in an area of 200 square miles around Pittsburgh, the company uses two clarification-softening systems, a conventional sedimentation plant and two upflow units.

Hagan Coagulant Aid #18, used in both systems, has produced these four main results:

1) REDUCTION OF CARRYOVER—Coagulant Aid #18 is particularly effective in cold winter months during low softening periods. Its effectiveness is evidenced by continuously good floc formation, despite reduced feed of softening chemicals.

2) EFFECT ON SLUDGE AND FLOC—the aid produces a tougher, heavier floc, stabilizing sludge blankets. This improves water clarification and reduces problems caused by carryover.

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4) BONUS BENEFITS—by stabilizing sludge in the clarifiers, Coagulant Aid reduces total alkalinity which, in turn, leads to substantial savings in required softening chemicals.

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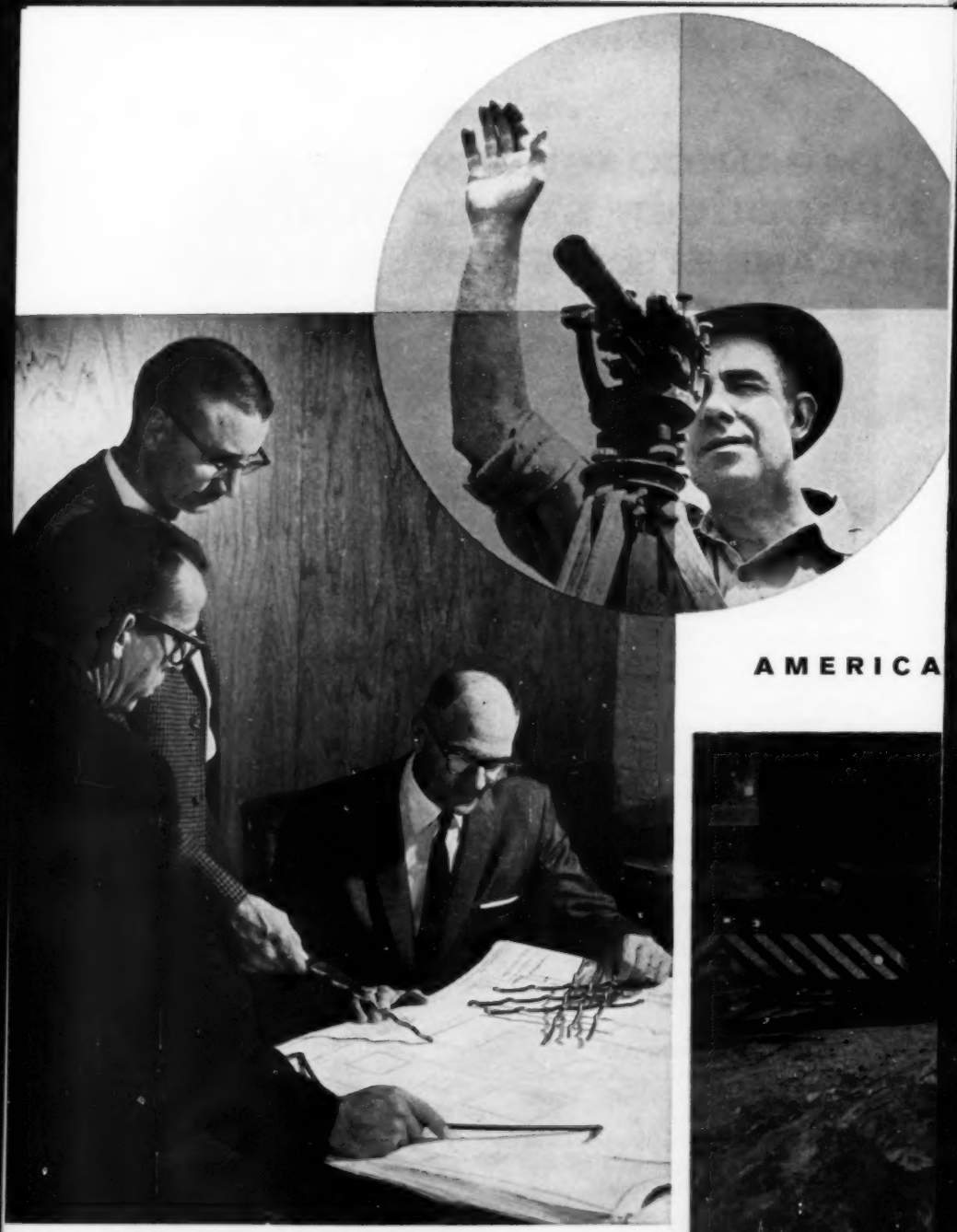

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Ion exchange is discussed in considerable detail in a new, 152-page manual, published as a technical reference work. After an introductory section on the general properties of ion-exchange resins, the book continues with descriptions of laboratory procedures and then a section devoted to applications of ion exchange. Requests for copies should be addressed to Irving M. Abrams, Chemical Process Co., 1901 Spring St., Redwood City, Calif.

Automatic analyzers for the continuous measurement of trace quantities of solids in liquids are described in a new 4-page bulletin. The information and illustrations are available in Bulletin WC-127, which may be obtained from Graver Water Conditioning Co., 216 W. 14th St., New York 11, N.Y.

Steel pipe is the subject of an illustrated, 8-page booklet that describes the diversified use of the pipe on several construction projects. Bulletin 1-P is available from Steel Plate Fabricators Assn., 105 W. Madison St., Chicago 2, Ill.

Check valves, double cushioned for protection against hammer and surge, are the subject of a 4-page illustrated bulletin, No. WS-1A. This may be obtained by writing Golden Anderson Valve Specialty Co., 1221 Ridge Ave., Pittsburgh 33, Pa.

Automatic feeders that feature a pneumatically operated belt are discussed in a 2-page data sheet, which includes detailed dimension drawings. Bulletin 36.20-1 is available from B-I-F Industries Inc., 345 Harris Ave., Providence, R.I.

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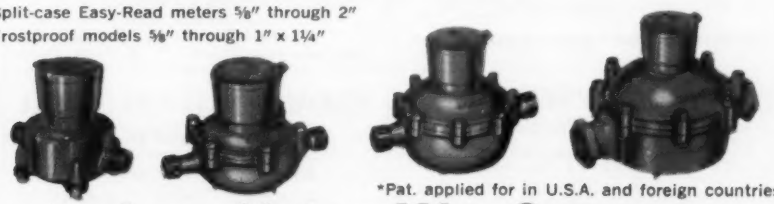

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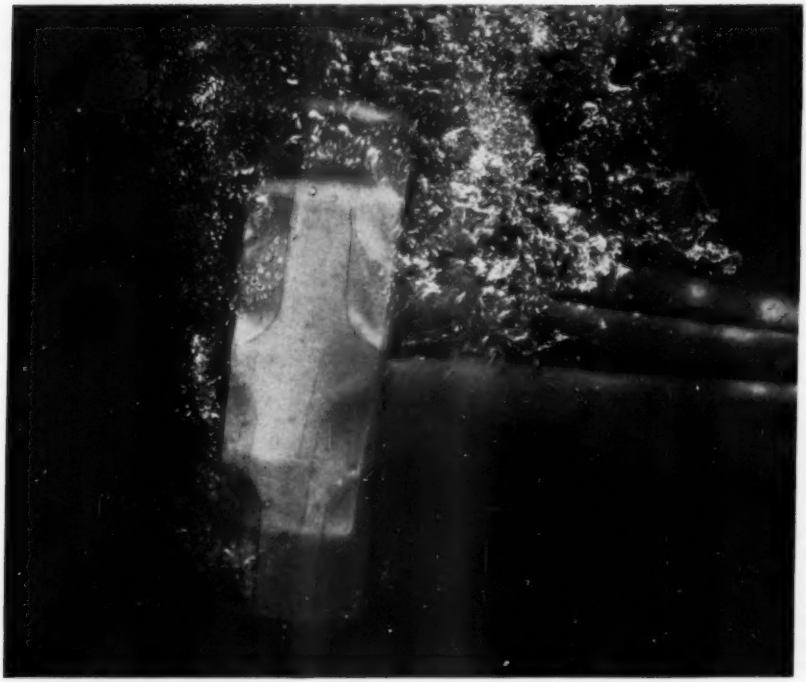

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24	69	335	1040	86	420	1300	102	505	1560	119	585	1820	136	665	2080
30	86	270	835	108	340	1040	129	405	1250	150	475	1455	170	535	1670
36	106	225	695	130	280	870	155	340	1040	181	395	1210	206	445	1390
42	119	195	595	148	240	745	177	285	890	206	335	1040	234	380	1190
48	137	170	520	171	210	650	203	250	780	236	290	910	268	330	1040
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96										470	145	455	539	165	520

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Key: In the reference to the publication in which the abstracted article appears, 39:473 (May '47) indicates volume 39, page 473, issue dated May 1947. If the publication is paged by the issue, 39:5:1 (May '47) indicates volume 39, number 5, page 1, issue dated May 1947. Abbreviations following an abstract indicate that it was taken, by permission, from one of the following periodicals: *BH—Bulletin of Hygiene (Great Britain); CA—Chemical Abstracts; Corr.—Corrosion; IM—Institute of Metals (Great Britain); NSA—Nuclear Science Abstracts; PHEA—Public Health Engineering Abstracts; SIW—Sewage and Industrial Wastes; WPA—Water Pollution Abstracts (Great Britain).*

CHEMICAL ANALYSIS

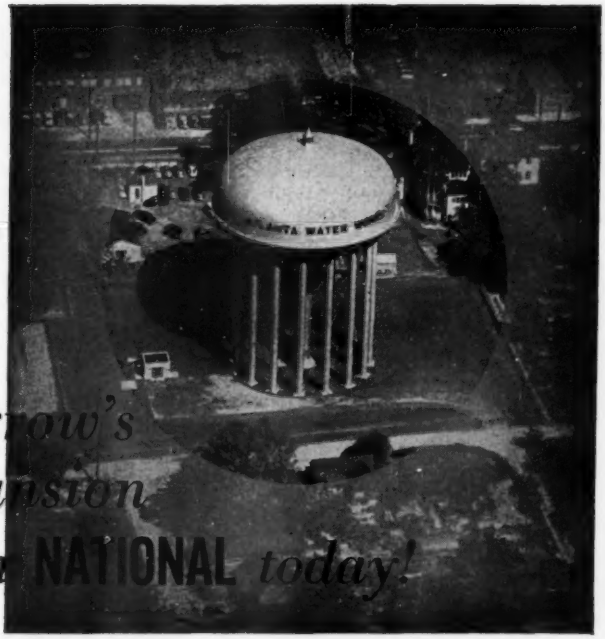
Determination of Microelements in Natural Waters. V. YA. EREMENKO. *Gidrokhim. Materialy* (USSR), 29:242 ('59). Adsorption of Co and Fe on the glass walls of green and white flasks and on filter paper is studied with the aid of radioactive tracers (Fe, Co). The samples of Don River water, contg. 640 mg/l or 130 mg/l of mineral substances, are studied; and the radioactivity is measured by the Goryunov method. The results show that the adsorption of microelements on the walls of flasks is negligible: 1.4–3.3% for white glass flasks, and 0.9–2.6% for green glass flasks. The dissolving of cations, contained in white and green glass, in doubly distd. water is studied. Al and Fe ions are found in water stored in green flasks, and Al ions only in that stored in white flasks. No other microelements are detected. Study of adsorption of microelements of Don River water, contg. 300 mg/l of mineral substances, on filter paper, shows that small quantities of Ni, Cu, Pb, Mn, and Fe are adsorbed on the filter. These quantities diminish after several filtrations have been made. During filtration, to avoid the loss of microelements contained in water, it is necessary to reject the first filtrates—for example, 1 liter of filtered water, when it contains 300 mg/l and 300 ml when it contains 760 mg/l and more of mineral substances. To avoid the adsorption of microelements on surfaces during the storage of natural waters in glass vessels, it is first necessary to wash the vessel twice with the water to be investigated.—*CA*

Use of Phenolate-Hypobromite Reaction in the Determination of Ammonia in Natural Waters. V. G. DATSKO & V. T. KAPLIN. *Gidrokhim. Materialy* (USSR), 29:230 ('59). A modified Thomas method is described in which hypobromite is used instead of hypochlorite. This substitution has many advantages. The optimal conditions of this reaction are studied: to 100 ml of natural water are added 2 ml NaOPh soln. and 15 ml NaBrO soln. at 20°C. The max. intensity of pure blue color is obtained after 40–50 min and is stable during 3–4 hr. More NaOPh is prepd. as follows: 1.0 g $Na_2S_2O_3 \cdot 5H_2O$ is dissolved in 100 ml H_2O. To this soln. 9.0 g NaOH, then 18.0 g PhOH, are added, and the mixt. is made up to 300 ml with water. The $Na_2S_2O_3$ is added as antioxidant to stabilize NaOPh; this soln. can to be kept 2 or 3 months. NaBrO soln. is prepd. as follows: to 100 ml of 9% NaOH soln. is added 1.5 ml of Br. The mixt. is carefully agitated in a dark glass vessel, then dild. with water to 300 ml. The soln. contains about 1.5% Br, and has a free NaOH concn. equal to about 0.6*N*. The NaBrO soln. can be used during 10–12 days, at 20–25°C. The degree of precision of the reaction, being equal to 5–10%, is similar to that of the Nessler reagent. Its sensitivity is about 0.01 mg/l of NH_4^+.—*CA*

Mercurimetric Determination of Chloride Content of Water Supplies. M. S. I. ACUNA & F. M. QUITAR. *Rev. obras sanit. nacion* (Buenos Aires) 41:216 ('59). The detn. of chloride in H_2O by titration with $Hg(NO_3)_2$ at pH 3.5 by using diphenylcarbazone as an indicator is compared with the classical method of Mohr with $AgNO_3$. For routine work the accuracy is equal to that of the method of Mohr, and it gives an endpoint in a clear soln. Where dilns. are required with the method of Mohr, the mercurimetric detn. gives more consistent results. The method can be used with concns. of 1–500 mg/l and with samples as small as 10 ml.—*CA*

Method of Concentrating Cu, Zn, Pb, Co, Hg, Ag, V, W, Mo by Precipitation With Sulfides and the Determination of the Microelements Adaptable to the Analysis

(Continued on page 64 P&R)



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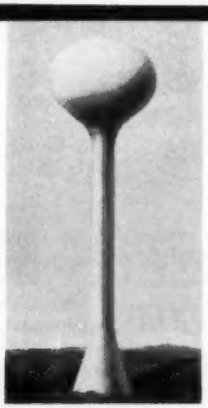
(*Continued from page 62 P&R*)

of Natural Waters. A. P. DEGTYARENKO; R. I. LIBINA; & A. D. MILLER. *Gidrokhim. Materialy* (USSR), 29:264 ('59). A short method was developed for concg. some of the microelements in natural waters by pptn. with CdS, Bi_2S_3, and ZnS. The simplicity of the method makes it useful under difficult expeditionary conditions. The pptn. of the microelements with CdS was as follows: 1 liter of water was combined with 5.5 ml 0.4*N* $CdCl_2$ soln., mixed, and 10 ml 0.4*N* Na_2S soln. added, again mixed, and in 2–3 min combined with 5.5 ml $CdCl_2$ soln. If the presence of V and Mo was suspected, a $FeCl_3$ soln. contg. 4–5 mg Fe was added (previous to the addn. of $CdCl_2$ and Na_2S). The ppt. was treated with aqua regia or with Br and HNO_3, the solvent evapd., dissolved in acid, and the amt. of microelements detd. For the Cu detn., the CdS ppt. was dissolved in concd. HCl and the soln. evapd. to dryness in a quartz evapg. vessel. The ppt. was treated with aqua regia and HCl, evapg. each time to dryness. The dry residue was dissolved in 10 ml 0.1*N* HCl, transferred to a separatory funnel, neutralized with NH_4OH (indicator thymol blue changed to greenish blue), then Cu extd. with diethylthiocarbamate, and detd. colorimetrically. The sensitivity for Cu is 0.5 γ/l. For the detn. of Zn, the CdS ppt. was treated with 10 ml aqua regia, the soln. evapd. to dryness, HCl added, again evapd., and then dissolved in 4–5 ml 0.1*N* HCl, 10–15 g KI added (to bind the Cd), and quant. transferred to a 25-ml volumetric flask. An aliquot was taken and the detn. of Zn was performed with dithizone. The sensitivity for Zn was 0.25 γ/l. For the V detn., the CdS ppt. was dissolved in HNO_3, a few drops of Br added, and the soln. evapd. to dryness. The residue was dissolved in 0.5*N* H_2SO_4 and V was detd. in form of the phosphotungstate complex. The sensitivity for V was 2–3 γ/l. For the detn. of Mo the CdS ppt. was dissolved in a min. amt. of HCl, the soln. evapd. to dryness, 3–4 ml of 0.5*N* NaOH and 2–3 ml 6% H_2O_2 added, and again evapd. The residue was dissolved in 7 ml. concd. HCl and Mo was detd. in form of the thiocomplex. The sensitivity for Mo was 0.5 γ/l. For the detn. of W the ppt. was treated with aqua regia, the soln. evapd. to dryness, the residue dissolved in HCl with addn. of a few drops of Br, and again evapd. to dryness. The residue moistened with tartaric acid and with 3–4 ml water was transferred to a colorimetric tube for detn. The sensitivity for W was 3 γ/l. For the detn. of Ag, the CdS ppt. was dissolved in 3 ml concd. HNO_3 and the vol. decreased by evapn.; then 10 ml *N* H_2SO_4 was added, evapd. till white vapors appeared, the walls of the dish were washed, and again evapd. to a moist salt. The residue was dissolved in 0.5*N* H_2SO_4 and Ag was detd. in form of a dithizonate by the mixed color with Cu dithizonate. The min. amt. Ag detd. was 0.5 γ/l. Hg was detd. in form of a dithizonate, but about 50% of the Hg volatilized. All the elements except Ni have a complete pptn. with the collector. The incompleteness of the Ni pptn. can be attributed to the production of colloidal solns. by NiS.—*CA*

Rapid Determination of Nitrates in Fresh Waters. MEI-KENG HSU; KEN-FANG WEN; & YUN-FENG JEN. *Hua Hsüeh Hsüeh Pao*, 24:329 ('58). The crit. conditions of a modified Nelson method applicable to the detn. of NO_3^- in water were studied. To 8 ml of sample in a 15-ml centrifuge tube, add 2 ml of AcOH soln. (made from 100 ml of AcOH, 1.65 ml of 0.2% $CuSO_4 \cdot 5H_2O$, and 1.35 ml of H_2O or the same amt. of 1% N_2H_4 (the latter to remove NO_2^- if it is present), boil 10 min, cool, add 0.6 g of pwd. reagent (made from 100 g of $BaSO_4$, 10 g of $MnSO_4 \cdot H_2O$, 2 g of Zn, 75 g of citric acid, 4 g *p*-$NH_2C_6H_4SO_3H$, and 2g of 1-$H_2NC_{10}H_7$), shake 1.5 min, centrifuge 10 min, and measure the transmittance of the supernatant at 515mμ. The method was stable, accurate, and rapid for a water sample contg. less than 0.02 ppm NO_3^-.—*CA*

Determination of Naphthenic Acids in Underground Waters. T. F. GUBINA. *Trudy, Vsesoyuz, Neftegaz. Nauch.-Isseldovatel. Inst.* (USSR), No. 14, 199 ('58). On storage of water samples, some or all of the naphthenic acids may ppt.; they ppt. completely at pH between 2 and 5, are partially dissolved at pH 5.8–6.0, and are completely dissolved at pH 6.8 and higher. Formation of ppts. of $CaCO_3$ and $Fe(OH)_3$ stimulates the pptn. The entire content of the flasks, including the ppt., should be taken for analysis. Neutralize to methyl orange with 0.1*N* HCl or NaOH, add 10 ml 0.2*N* $NaHCO_3$ for

(*Continued on page 66 P&R*)



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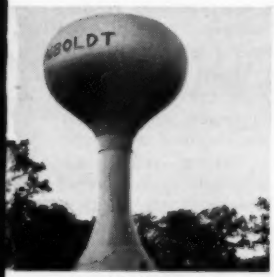

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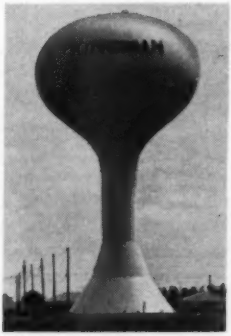

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(Continued from page 64 P&R)

every 0.5 liter water, then 10 ml ligroine or $CHCl_3$, remove the colored solvent, and repeat the extn. until the solvent is colorless. Decant the water layer, contg. the naphthenic acids, acidify with 3*N* HCl to pH 3–5, add freshly prepd. $Fe(OH)_3$, mix well, let settle, decant, and det. the naphthenic acids in the residue by the Orlov method.—*CA*

Colorimetric Determination of Zn in H_2O Using Brilliant Green and Thiocyanate. G. U. HOUGHTON. *Proc. Soc. Water Treatment Exam.*, 6:60 ('57). A method is described for the detn. of Zn in drinking waters. It is based on the reaction of the water with Brilliant Green in an acid soln. contg. thiocyanate. Details of the test and the reagents required are given. The sensitivity obtained was such that 5 cc of sample permitted detn. of Zn over the range 0.5–3 ppm. Al and Pb (10 ppm) do not interfere, and up to 3 ppm Cu and Fe may be present without effect on the detn. In 12 check analyses the accuracy was within 8%.—*CA*

Methods for the Investigation of Physical Properties and Chemical Composition of Fresh Water. O. A. ALEKIN. *Zhizn Presnykh Vod S.S.S.R.*, 4:243 ('59). The vols. of samples required for complete analysis of fresh water (from rivers and lakes) were specified, and the preliminary treatment of the bottled samples was described. The examn. of phys. properties involved color detn. by the K_2PtCl_6–$CoCl_2 \cdot 6H_2O$ method and estn. of clarity by the typescript reading test. Total org. substances in fresh water were estd. by oxidizability tests with either $KmnO_4$ or $K_2Cr_2O_7$. Total Fe and Fe^{+++} were detd. by KCNS, and Fe^{++} was calcd. NH_3 was estd. colorimetrically with K_2HgI_4; the solns. used and tints obtained were tabulated. The concns. of nitrites and nitrates were detd. by α-naphthylamine and diphenylamine, resp. The mineral and org. forms of the P compds. present in fresh water were distinguished. HCO_3^- was detd. by the $Na_2B_4O_7$ method, and the $H_8Si_2(Mo_2O_7)_6$ complex was utilized for the Si detn. The total hardness of fresh water was estd. by K

(Continued on page 68 P&R)

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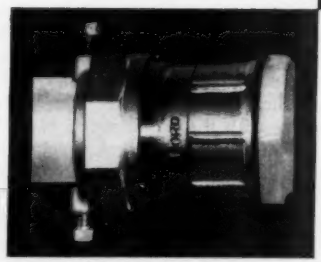

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(*Continued from page 66 P&R*)

palmitate (0.1*N*) titration, while K + Ca was detd. by a modified EDTA method. Ca, Cl, and SO_4 ions were detd. by standard lab. methods. The Mg concn. was calcd. from the total hardness less the Ca content. Alkali metal ions and the total ion compn. of water were calcd. A method is given for detg. $CaCO_3$ satn. of water. Changes in chem. compn. of water over its depth and width, in daily, monthly, and yearly intervals, up and down rivers are given.—*CA*

Determination of Iodine in Water. M. T. Golubeva. *Inform. Metod. Materialy Gosudarst. Nauch.-Issledovatel. Sanit. Inst.* (USSR), No. 3, 32 ('55). For the detn. of I^-, 1–3 liters of H_2O, siphoned off from the ppt., is treated with 50% K_2CO_3 (250 g of the salt is dissolved in 200 ml H_2O, shaken 1st with 25 ml EtOH and, after removal of the alc. layer, with 100 ml EtOH). The shaking is repeated 5–10 times, the aq. soln. is evapd., and the crystals are filtered off to an alk. reaction against phenolphthalein. The soln. is evapd. to dryness. The org. matter is removed by heating at about 400°C; the residue is wetted with 2–3 drops of H_2O and heated to dryness. The residue is treated with 5–10 ml H_2O, filtered, and the filtrate is evapd. The residue is made into a paste with 2 drops of H_2O (or 1–2 drops of satd. K_2CO_3). I is extd. from the paste by 5–6 portions (3–5 ml each) of EtOH; the alc. soln. is made alk. with 1–2 drops of 5% K_2CO_3 and evapd. The residue is treated with 3, 1, and 1 ml of H_2O. The soln. is acidified by 0.5*N* H_2SO_4 by use of methyl orange paper indicator; 0.5 ml excess acid, pumice, and 3 drops of fresh Br soln. are added and the soln. is boiled for 5 min. KI soln. is added and the sepd. I is titrated with 0.001*N* $Na_2S_2O_3$.—*CA*

Determination of Micro Amounts of Dissolved Oxygen in Water. M. Needleman. *Analyst*, 84:720 ('59). The detn. of dissolved O in the range 0.0–0.050 ppm is reported. The app. used incorporates certain novel features which allow the reagents to be rendered O-free before the detn. and to be introduced through separate entries. The use of the app. and its standardization is described. The chem. procedure followed is that of the ASTM. The app. is primarily for laboratory use, but should be capable of adaptation for analysis of boiler water.—*CA*

Determination of Radioactive Strontium in Open Water Reservoirs. V. L. Zolotavin & L. K. Ponomareva. *Radiokhimiya* (USSR), 2:104 ('60). By means of the rhodizonate method of Sr sepn. (as insol. Sr rhodizonate at pH = 6–7) from Ca, standard HCl-solns. of $Sr^{90} + Y^{90}$ without carrier were prepd. The exact procedure of Sr detn. in natural water reservoirs is described in detail. Required time is 1.5–2 hr; max. relative error ± 10%. Sensitivity of the method is 5×10^{-11} c (that is, 1 order of magnitude higher than the permissive limit of Sr^{90} in water). Cs^{137}, Ru^{106}, Ce^{144}, and Zr^{95} do not interfere.—*CA*

Analysis of Water With the Use of the Cation Exchanger KU-2. L. N. Podgornyi. *Zavodskaya Lab.* (USSR), 26:717 ('60). The sulfopolystyrene cation exchanger KU-2 is suitable for conducting ion-exchange detns. in water analysis. Salts of org. acids increase the results of ion-exchange detns. of the sum of anions. In waters contg. little or no nitrate, nitrite, phosphate, and org. acid anions, the results of the trilonometric detn. of sulfates agree well with those obtained by cation-exchange. The following scheme is proposed for the analysis of waters with a mineralization of 0.05–1.5 g/l. The pH is detd. colorimetrically with phenol red or universal indicator. The HCO_3^- ions are detd. by titration of the water sample with HCl in the presence of methyl orange or a mixed indicator (methyl red and methylene blue). If CO_3^{--} ions are present the sample is first titrated to the phenolphthalein endpoint. The sum of the anions is detd. by titration with alkali or borax of acidity formed during H^+ cationization of the water tested, following the preliminary titration of HCO_3^-. The sum of the strong acids is detd. by titration with alkali or borax of the acidity formed during H^+-cationization of water not previously neutralized. Cl^- is detd. by the Mohr method. The SO_4^{--} content is calcd. as the difference between the sum of the strong acids and the Cl^- concn. The SO_4^{--} content may also be detd. trilonometrically in the cationized water. The hardness is detd. trilonometrically in the presence of acid chrome dark blue.—*CA*

(*Continued on page 70 P&R*)

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Colorimetric Micro-Determination of Copper in Water: A Survey of Available Methods. B. TUCK & E. M. OSBORN. *Analyst*, 85:105 ('60). As the Feed Water Standards Panel of the Central Electricity Generating Board has recommended that, for boilers operating at or above 1,500 psi, the concn. of copper and iron in feedwaters should together not exceed 10 μg/l, known anal. techniques were surveyed to establish a method permitting rapid and accurate detn. of these impurities at these low concn. Of the five colorimetric methods examined for the detn. of copper, the sodium diethyldithiocarbamate method lacks sensitivity and selectivity; the bis*cyclo*hexanone oxalyldihydrazone and oxalyldihydrazide methods are simple to carry out and give satisfactory results, but necessitate preliminary concn. by an ion-exchange technique; and the neocuproine method is satisfactory but has been superseded by the 2,2′-diquinolyl method which gives a superior color, simpler technique, and excellent reproducibility near the zero point of the calibration curve. It is unaffected by the presence of other substances encountered in boiler feedwater, such as iron, nickel, zinc, hydrazine, and volatile amines (such as *cyclo*hexylamine and morpholine).—*WPA*

Complexometric Titration Method for Determining Calcium in the Presence of Magnesium. A. RINGBOM; G. PENSAR; & E. WANNINEN. *Anal. Chim. Acta*, 19:525 ('58). A complexometric method for the detn. of calcium in the presence of magnesium is described. Calcium is titrated with ethyleneglycol-bis-aminoethylethertetra-acetic acid (EGTA), using Zincon and ZnX (a zinc compound) as indirect indicator. Details of procedure are given. The stability constants of the zinc complexes of EGTA and Zincon were detd. and suitable experimental conditions are suggested on the basis of theoretical considerations. The results of some titrations are presented in a table.—*WPA*

(*Continued on page 72 P&R*)

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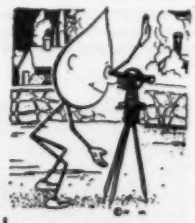

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(*Continued from page 70 P&R*)

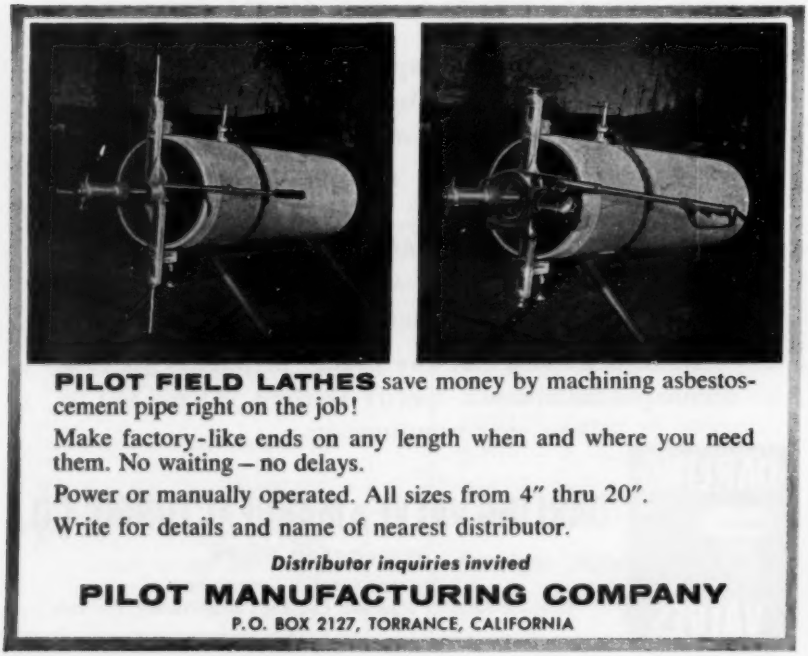
Determination of Hardness-Forming Substances in Water. A.-G. RURCHEMIE; H. LIST; A. GUBIN; & M. JUNG. *Chem. Zbl.*, 128:9187 ('57). To det. hardness-forming substances in water, measured amts. of water are mixed in succession with measured amts. of an indicator soln. and of a buffer soln. The mixtures are passed through a bulb through which light is passed to a photo-cell which converts it to energy impulses used for operation of measuring, indicating, and controlling app. Indicators and buffer solns. used are within a range of 0–0.2° (German) of hardness.—*WPA*

Determination of Fluoride by Conductometric Titration. H. KUBOTA & J. G. SURAK. *Anal. Chem.*, 31:283 ('59). A method for the volumetric semimicro detn. of fluoride has been devised, based on the reaction of fluoride with lanthanum acetate and using conductometric detection of the end-point. The procedure is best applied to the titration of distillates from the Willard-Winter sepn. A modified semimicro-distillation app. is described, in which the sample is distd. at const. temp. and nearly const. vol., to minimize the amt. of extraneous ions being distd. as well as the volume of the distillate. A relative standard deviation of 1.8% for the distn. and titration was obtained from replicate detns. of 1-ml aliquots of a standard fluoride soln. containing about 5 mg of fluoride.—*WPA*

Rapid Determination of NO_2 and NO_3 in Natural Waters. TING-YUAN HOU. *Shu Wen Ti Chih Kung Ch'eng Ti Chih* (China), 2:31 ('59). The recommended procedure is simple in manipulation and is adapted to field analysis. NO_3^- is 1st reduced by Zn to NO_2^-, the latter gives a pink color with *p*-nitrobenzenesulfonic acid and 1-naphthylamine. The color is compared with standard blanks.—*CA*

Oxidized Nitrogen in Waters and Sewage Effluents Observed by Ultra-Violet Spectrophotometry. R. C. HOATHER & R. F. RACKHAM. *Analyst*, 84:548 ('59). Measurement of the ultraviolet absorption at

(*Continued on page 74 P&R*)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(*Continued from page 72 P&R*)

2,100Å affords a rapid method for detg. nitrate. For most waters of good org. quality, its sensitivity and accuracy compare favorably with the usual chemical methods, and results are in agreement with such methods. However, for some samples containing nitrate freshly formed by microbiologic action, there are discrepancies suggesting the presence of unstable compounds that change to nitrate when kept. The use of a spectrophotometer is therefore suggested as an additional method for investigating nitrification in soils, waters, and sewage effluents.—*PHEA*

Determination of Alkylbenzenesulphonates in River Waters and Sewage. H. L. Webster & J. Halliday. *Analyst*, 84:552 ('59). An improved colorimetric method has been developed for the accurate detn. of alkylbenzenesulphonates in waters. The procedure is less subject to error from naturally occurring substances than is Longwell and Maniece's method. Agreement with infrared analysis is good, and the method is of value when infrared equip. is not available or when a more rapid result is required. The method can also be used satisfactorily on sewage effluent and settled sewage when more accurate detergent analysis of these is needed. The method is simple to perform and combines principles from other reported procedures; it gives good recoveries of known detergent additions and is reproducible.—*PHEA*

Formate Lactose Glutamate: A Chemically Defined Medium as a Possible Substitute for MacConkey Broth in the Presumptive Coliform Examination of Water. R. D. Gray. *J. Hyg.*, 57:249 ('59). During a period of 13 months, 1,273 consecutive samples of water received at this lab. were submitted to a modified presumptive coliform test which included single tubes of four different media. These were: (1) MacConkey broth; (2) formate lactose glutamate medium (pH 7.5); (3) lactose glutamate medium (pH 7.5); and (4) Folpmers's glucose glutamic acid (pH 6.0). A total of 955 samples gave exactly the same results in all four media, but the remaining 318 samples produced differences which enabled comparisons to be made between the media.

As compared with MacConkey broth, the three glutamic-acid media produced 24–30% more isolations of *Esch. coli*, and the formate and the glucose media produced at least the same total number of true coliform organisms (including *Esch. coli*) with appreciably fewer false positive reactions. The lactose glutamate medium gave no false positive reactions but, through the suppression of coliform organisms other than *Esch. coli*, reduced by 16 the total coliform yield. MacConkey broth gave the largest early (18 and 25 hr) yield of positive reactions, but the result at the end of 24 hr showed the formate medium to be not far behind MacConkey broth and appreciably ahead of the glucose medium.

Further expts. with the formate lactose glutamate medium adjusted to different pH's, ranging from 6.0 to 7.5, indicated that a medium of pH 6.7 provided optimal conditions for the early development of both acid and gas. It was therefore decided to test this observation more fully with routine samples. During a period of 4 months, 279 unselected consecutive samples of water (of which 78 gave positive results) and 57 consecutive unchlorinated water samples (56 of which gave positive results) were examined by a multiple-tube technique in both MacConkey broth and formate lactose glutamate medium (pH 6.7). In the former series the MacConkey broth contained the inhibitory bile salt previously used; in the latter series this was replaced by a relatively noninhibitory bile salt. In both series the formate medium yielded an appreciably increased total of coliform organisms (12 and 17%) including an increased total of *Esch. coli* (27% in the former and 13% in the latter series), and in both series also the results at 18 hr were abreast with, and at 24 hr ahead of, those obtained in MacConkey broth.

With certain reservations, it is considered that formate lactose glutamate medium (pH 6.7) can be offered as a suitable alternative to MacConkey broth for the presumptive coliform test of water. Wider trials of this medium would be required before this claim could be fully substantiated.—*PHEA*

CORROSION

Influence of Corrosion Upon the Quality of Drinking Water. L. Rott et al. *Orv. Szle* (Hung.), 4:333 ('58). The central water supply installation of a town was studied. The purification station of the installation treated river water of a low "hardness" by

(*Continued on page 76 P&R*)

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(Continued from page 74 P&R)

means of coagulation with aluminum sulfate. The resulting drinking water was transported to the town through a 14-km pipeline. The quality of the water on leaving the purification station was acceptable, but at the end of the pipeline it was turbid and yellowish and had an unpleasant taste. The deterioration of organoleptic properties was due to corrosion within the pipeline, evidence of which are the negative value of the stability index, the high concn. of divalent carbonic acid, and the increase in iron concn. during transport.—*PHEA.*

Visual Study of the Corrosion of Defected Zircaloy-2-Clad [Uranium] Fuel Specimens by Hot Water. E. F. STEPHAN; P. D. MILLER; & F. M. FINK. *US At. Energy Comm. Rept.*, BMI-1387 ('59). The failure of defected Zircaloy-2-clad U and U-2 wt.-% Zr fuel specimens in high-purity, high-pressure water at 200–345°C was investigated. Tests were carried out in a windowed autoclave and time-lapse motion pictures were taken to record the progressive changes ending in the complete disintegration of the core materials in the specimens. When clouding of the water by corrosion products makes visual observation impossible, the progress of the reaction is monitored by continuous measurement of the pressure increase caused by H accumulation, a result of the reaction $U + 2H_2O \rightarrow UO_2 + 2H_2$. The nature of the attack was similar for all specimens. The slow formation of a blister in the cladding area surrounding the defect was followed by a copious evolution of H at the base of the swollen area. In general, a crack was visible in the cladding at this stage. Catastrophic failure of the specimen followed. The time required for each phase of the reaction $\propto$ temp. Initial swelling occurred after $\sim$ 24 min at 345°C, but only after 8 hr at 200°C. Diffusion-treated U–2 wt.-% Zr-cored specimens were most resistant to attack. Specimens with β-treated, water-quenched, natural-U cores were less resistant.—*IM*

(Continued on page 78 P&R)

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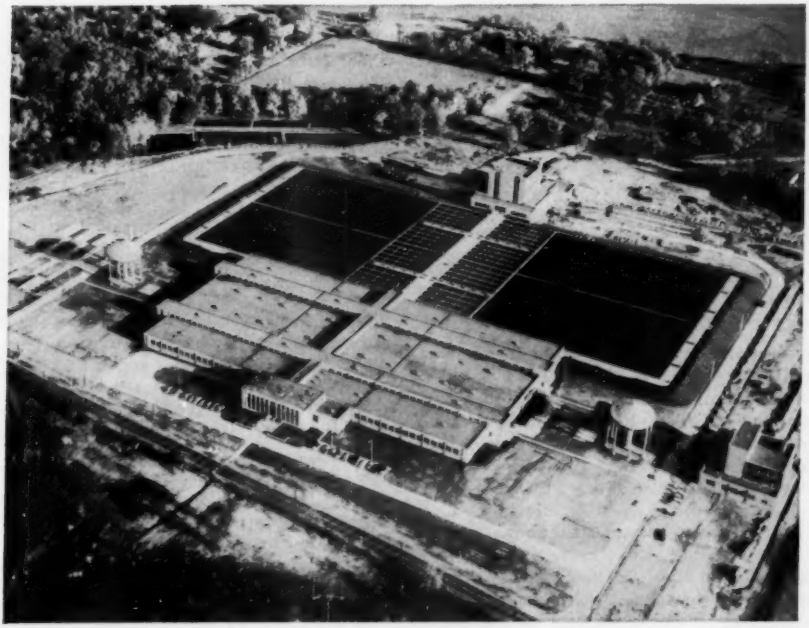

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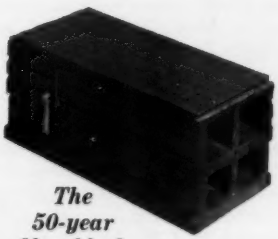

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(Continued from page 76 P&R)

Removal of Rust and Scale From Steam Generators. E. J. SEYB. *Brennstoff-Warme-Kraft* (Ger.), 12:9 ('60). The following methods are compared on the basis of experience: boiling with Na triphosphate soln.; pickling with dil. acids and hydrazine-HCl soln.; boiling with solns. of ethylenediaminetetraacetate ("Komplex-All"). The latter dissolves Ca, Mg, and Fe deposits by chelation. It was applied at a concn. of 1 g-mole/l and was highly effective and noncorrosive to metal.—*CA*

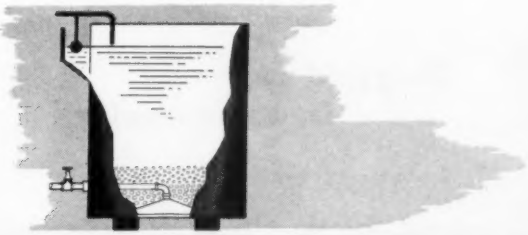
Aluminum Alloys for Water-Cooled Power Reactors. E. C. W. PERRYMAN. *J. Inst. Metals*, 88:62 ('59–'60). The corrosion properties of Al–Ni–Fe alloys in both static and dynamic high-temp. water are described. Corrosion-resistance in high-temp. water flowing at 20 fps is ~ 8 times that in a static system, and there is little effect of alloy compn. under dynamic conditions. Under flowing conditions, the corrosion rate is very dependent on the amount of Al surface within the dynamic system, and this is thought to be due to saturating the water with Al_2O_3. Under dynamic conditions, the lowest corrosion rate observed is $\sim 8 \times 10^{-4}$ in./year, but an oxide film formed on samples in static water will reduce the corrosion rate to $\sim 2 \times 10^{-3}$ in./year. The effect of inhibitors is discussed, and also that of heat flux. Results are given for both short- and long-term mech. properties. The properties of these alloys are discussed in relation to water-cooled power reactors; at the present stage of development they are not competitive with Zr alloys for fuel sheathing.—*IM*

Corrosion Inhibitors for Cast Iron and Other Metals in Ethylene Glycol Solutions and Mains Water. A. D. MERCER & F. WORMELL. *J. Appl. Chem.*, 9:577 ('59). Previous studies were extended to the protection of cast-Fe, Cu, Al, and composite specimens by $C_6H_5COONa + NaNO_2$ in unheated systems. Higher inhibitor concn. were necessary in these conditions, but effective protection in 25% ethylene glycol was obtained with 5% $C_6H_5COONa + 0.3\%$ $NaNO_2$; this also gave protection in intermittently heated soln., though 1.5% benzoate + 0.1% nitrite was equally effective. Without inhititors, corrosion of some of the metals (not Cu) was slower in the glycol soln. than in water. Electrode-potential detn. on metals in inhibited soln. at room temp. reflected relative corrosion behaviors.—*IM*

Corrosion of Inconel in 600°F Static Water. G. E. GALONIAN & H. L. TYMCHYN. *US At. Energy Comm. Rept.*, KAPL-2047 ('59). Static autoclave tests were made to det. the corrosion rate of Inconel in various waters at 600°F (315°C) and its resistance to intergranular corrosion and stress-corrosion cracking. Tests of Inconel-132 welds and a bimetal weld-cladding joint between Inconel and type 308 stainless steel were also made. Inconel possesses a low rate of metal loss in static de-aerated water, and an adherent film forms on it; 0.025% C appears to give resistance to intergranular corrosion and stress-corrosion cracking. A min. amount of corrosion occurs in water contg. dissolved H. Increasing pH to 9.5 with NH_4OH decreases corrosion of annealed Inconel in H-free de-aerated water. A stabilizing heat treatment may reduce corrosion slightly, compared with an annealing or sensitizing heat treatment. The welds and bimetal weld cladding were resistant to corrosive attack in the water conditions studied. Surface prepn. of test samples affects loose-deposit formation in H-free water. —*IM*

Inhibition of the Corrosion of Iron in Neutral and Alkaline Solutions. P. HANCOCK & J. E. O. MAYNE. *J. App. Chem.* 9: 352 ('59). Potentials at which iron is converted into anhydrous cubic oxide in certain inhibitive sols. were measured, and it is shown that these potentials are more negative than potentials acquired by unpolarized specimens immersed in solns. for 3 days. It is suggested that when iron, carrying its air-formed film, is placed in an inhibitive soln. saturated with air, corrosion current is concd. at weak spots in oxide film. At these points potential is consequently raised to that at which anhydrous cubic oxide is produced, air-formed film is thereby thickened, and corrosion prevented. When pH of inhibitive soln. exceeds 9, anhydrous cubic oxide may also be formed by interaction of anodic product and ferrous hydroxide, with atmospheric oxygen. This explanation of inhibition has been extended to account for corrosiveness of pure water, effect of concn. of inhibitor, and dual role of oxygen.—*Corr.*

(Continued on page 82 P&R)



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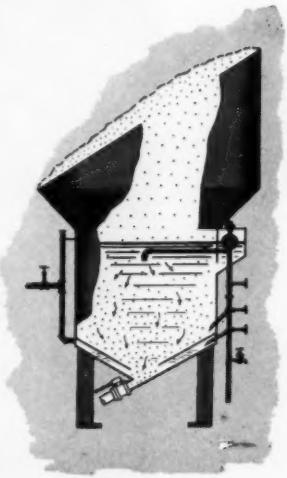

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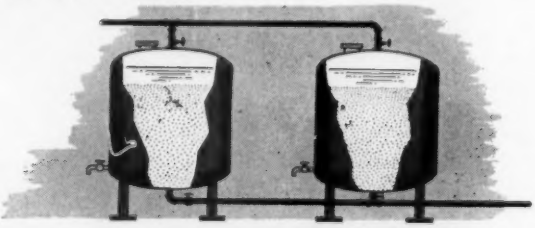

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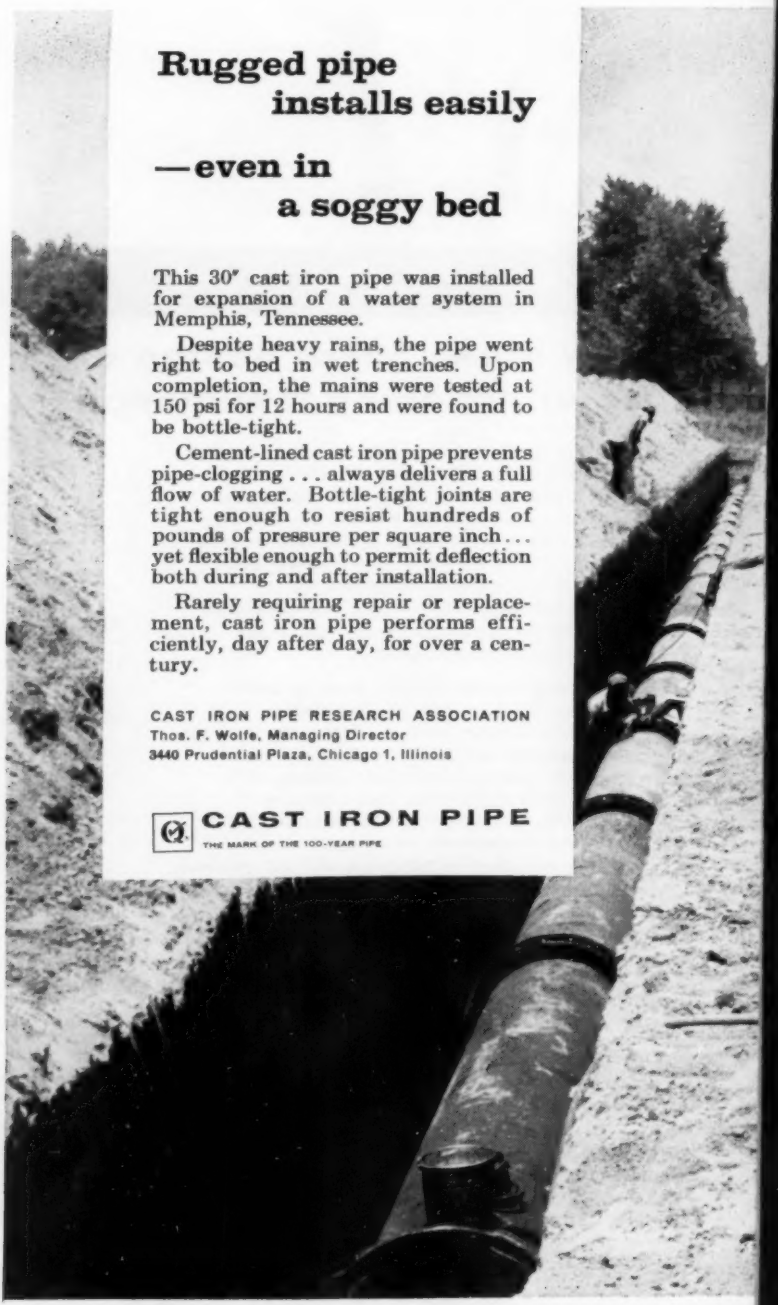

(*Continued from page 78 P&R*)

RADIOACTIVITY

Radiochemical Determination of Radium-B and Thorium-B in Radioactive Springs. I. HATAYE. *Mem. Fac. Sci., Kyushu Univ., Ser. C,* 3:71 ('58). The method was designed for detns. of radium-B (Pb^{214}) (I) and thorium-B (P^{212}) (II) in radon-containing carbonated mineral springs; I and II were extracted by a soln. of dithizone in CCl_4 (50 mg/l) from an ammoniacal 5% KCN soln. at pH 8–9. To prevent interference by Po and Bi, two elimination methods were proposed. It was found that Pb is extracted preferentially from a soln. contg. isotopes of Pb and Po when microgram quantities of inactive Pb are added to the soln. Bismuth was removed by washing the extracts with 0.5% KCN soln. after Pb and Bi had been extracted together. The respective activities of extracted I and II were measured by a Lauritsen-type electroscope; II was used as a tracer in the determination of I.—*PHEA*

Radon Content of Cold Waters From Certain Regions in France. M. ROUBAULT; R. COPPENS; & G. JURAIN. *Compt. Rend.,* 248: 715 ('59). By using a scintillation detector for α-rays in a mobile lab., the Rn content of 344 water samples from springs and wells in certain regions of France was measured in the summer of 1958. Except in mountainous districts, Rn was present in quantities greater than the recognized tolerance for continuous intake, 40×10^{-10} c/l, in more than $\frac{2}{3}$ of the samples, regardless of the presence or absence of U in the vicinity. The lower Rn contents of waters from mountainous areas are ascribed to loss of the element from these rapidly flowing streams. The authors question the value of the recognized tolerance figure for a continuous dose of radon, since the waters examd. had been used for humans and animals for very long periods.—*PHEA*

Radioactivity and the Distribution of Water From Lake Constance. J. HOLLUTA.

(*Continued on page 84 P&R*)

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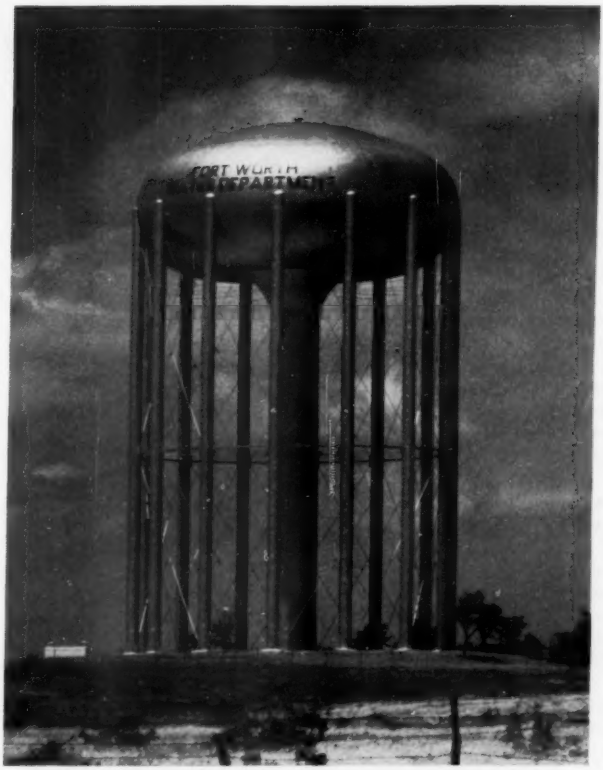

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(Continued from page 82 P&R)

Monatsbull. Schweiz. Ver. Gas- u. Wasserfachm., 38:102, 127 ('58). After a general discussion of the increasing use of radioactive substances and the causes and effects of radioactive contamn. of ground and surface waters, the radioactivity of Lake Constance is considered. The radioactivity of water in the lake near Riet, 40 m below the surface, increased tenfold, from 0.02 to 0.22×10^{-10} c/l in the period May 1956–Aug. 1957. In most cases, water taken from the lake for public supplies is only disinfected, and the treatment plants are incapable of reducing the radioactivity of the water. Even when the water is filtered, some substances, such as strontium-90, remain almost unchanged. The treatment of radioactive waste waters is also discussed. It is emphasized that any atomic plants to be built in the area should treat their waste waters to reduce the concn. of radioactive substances below the danger level, and provide auxiliary protection for use in the case of accidents; the storage of radioactive wastes in the basin of Lake Constance should be prohibited.—*PHEA*

Dependence of Artificial Radioactivity in Rain on Rainfall Rate. J. F. BLEICHRODT ET AL. *Tellus*, 11:404 ('59). The artificial radioactivity in rain was measured as a function of rainfall rate. As a rule an inverse relationship was found between the specific activity and the rate of rainfall. A tentative explanation is given.—*PHEA*

Protection of Drinking Water From Contamination With Radioactive Substances. A. N. MAREI. *Gig. i San.* (USSR), 9:7 ('56). The author stresses the need for amendment of the existing MPC of radioactive substances in drinking water reservoirs. The accumulation of such substances in water systems opens up the possibility of their transmission to the food supplies. Alluvial deposits may also be a source of secondary poln. of the water. It is essential to maintain continuous radiometric observation of open water reservoirs liable to radioactive poln. and to keep a close watch on the disposal of industrial effluents and their clearance. It is important to ascertain not only the abs. concn. of the radioactive elements in the water but also changes in their concn. An increase in the degree of poln. may serve as a warning that the sanitary installations are unsatisfactory. The control of radioactive poln. should include also the examn. of plants, molluscs, and fish.—*PHEA*

Radioassay of Uranium and Plutonium in Vegetation, Soil, and Water. E. L. GEIGER. *Health Physics*, 1:405 ('59). A method is discussed for the sepn. of uranium and plutonium from vegetation, soil, and water. The method is based on the extraction of uranium and plutonium from 4 to 6*N* nitric acid into 50% tri-*n*-butyl phosphate in *n*-tetradecane diluent. Uranium and plutonium are recovered together with sufficient reduction in total solids to allow α-counting and pulse height analysis. Data from several hundred "spiked" samples to which uranium and plutonium were added indicate a nearly equal recovery of uranium and plutonium. Avg recoveries are $76 \pm 14\%$ for vegetation, $76 \pm 16\%$ for soil, and $82 \pm 15\%$ for water. The procedure is designed for samples that may be collected and analyzed for radioactivity as a part of a health physics regional monitoring program.—*PHEA*

Problem of Radioactivity and Water Supply and Sanitary Protection of Water Sources. M. K. MARKARYANTS & E. V. SHTANNIKOV. *Tez. Doklady XIII Sezda Gig., Epid., Mikrob. I Infekts.* (Moscow), 1:55 ('56). Changes in the background radioactivity of water sources result from the widespread use of isotopes in the national economy. Self-purification of surface waters mainly depends on pptn. and adsorption on clay and other suspended particles, but also on removal of radioactive isotopes by bioplankton. For removal of radioactive substances, physical (dilution, deposition, storage, or distillation), chemical (coagulation, adsorption, ion exchange), and biologic methods can be used. The data obtained indicate the effectivity of coagulation by means of aluminum sulphate (in alkaline medium) in removal of P^{32} and I^{131}. Satisfactory deactivation of P^{32}, I^{131}, and Ca^{45} was achieved by means of fractional and, subsequently, total filtration using cation-exchange resin espatit-1 and anion-exchange resin EDE-10. The speed of filtration, the height of the column of the resins, and pH of medium should be considered. Filtration through river sand has little effect on radioisotope removal.—*PHEA*

(Continued on page 86 P&R)

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(*Continued from page 84 P&R*)

Normal Radium Content and the Ra^{226}-Ca Ratio of Various Foods, Drinking Water, and Different Organs and Tissues of the Human Body. H. MUTH ET AL. *Health Physics*, 2:239 ('60). New exptl. results concerning the normal radiation (Ra^{226}) content of food, drinking water, the distribution of normal radium in the human body, and the Ra^{226}/Ca ratio are reported. The daily uptake of radium by a "standard man" amounts to about 3×10^{-12} c, and under normal environmental conditions, 90% of this amount enters the body with food and only 10% with water. The natural radium is not concentrated exclusively in bone. Considerable amounts are distributed throughout the soft tissue and the different organs. Animal expts. (chicken) demonstrated that there exists no dependence on age of the normal radium content. The consequences of these results for the estimation of the distribution pattern of other bone-seekers are discussed.—*PHEA*

The Soil Ecosystem and Radioactive Waste Disposal to the Ground. S. I. AUERBACH. *Ecology*, 39:522 ('58). The possibility of using the ion-exchange capacity of soil to dispose of large volumes of radioactive waste waters is under investigation, but detailed studies are necessary to determine the effect of radioactive wastes on the soil ecosystem, which includes mineral particles, plant and animal residues, living systems, free and bound water, and gases. Preliminary studies have been carried out on the effect of gamma radiation from a cobalt-60 source on the substrate from a hollow tree, containing dense and complex arthropod populations. The results indicated that related species can differ considerably in their sensitivity to radiation.—*WPA*

Determination of Radioactivity in Saline Waters. V. J. SODD; A. S. GOLDIN; & R. J. VELTEN. *Anal. Chem.*, 32:25 ('60). A screening method for the detn. of gross radioactivity in saline waters is based on the pptn. of many cations as sulfides from ammoniacal soln. Carrier lanthanum, iron(III), cobalt(II), zinc, and nickel are pptd. while the major sea water salts are left in soln. An alkaline-earth fraction containing strontium, barium, and radium can be recovered by pptn. of carbonates from the sulfide filtrate followed by pptn. of strontium carrier as nitrate. Cesium can be recovered separately by pptn. with silicotungstic acid and conventional purification.—*PHEA*

Determination of Artificial Radioactive Isotopes in Water. A. A. BYLINKINA. *Lab. Delo* (USSR), 4:41 ('58). General details of a technique have been described to show how K^{40} can be applied as a standard for the measurements of β-emission of dried residues obtained after the evapn. of water samples. In this way the radioactivity of artificial radioactive isotopes present in such residues can be detd. An order of sensitivity of 10^{-13} c/l is claimed, which is less than the permissible value of radioactivity in water.—*PHEA*

Radioactivity and Drinkable Water. A. LAFONTAINE. *Tech. l'eau* (Brussels) 13: No. 148, 13; No. 149, 31 ('59). The action of ionizing radiations on the human body (somatologic and genetic effects), the nature of dangers encountered with drinking water contamd. by radioactive compds., the origin of possible contamns., and the measures used to avoid them in peacetime and in case of nuclear explosions are successively reviewed.—*CA*

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STEEL PLATE FABRICATORS ASSOCIATION
105 West Madison Street • Chicago 2, Illinois

STEEL PLATE FABRICATORS ASSOCIATION Founded 1933

Index of Advertisers' Products

Activated Carbon:
Industrial Chemical Sales Div. Permutit Co.

Activated Silica Generators:
B-I-F Industries, Inc.—Omega
Wallace & Tiernan Inc.

Aerators (Air Diffusers):
American Well Works
Carborundum Co.
Eimco Corp., The
General Filter Co.
Permutit Co.
Walker Process Equipment, Inc.

Air Compressors:
Allis-Chalmers Mfg. Co.
Worthington Corp.

Alum (Sulfate of Alumina):
American Cyanamid Co., Process Chemicals Dept.
General Chemical Div., Allied Chemical Corp.

Ammonia, Anhydrous:
General Chemical Div., Allied Chemical Corp.
Jones Chemicals, Inc.

Ammoniators:
B-I-F Industries, Inc.—Proportioneers
Fischer & Porter Co.
Wallace & Tiernan Co., Inc.

Ammonium Silicofluoride:
American Agricultural Chemical Co.

Associations, Trade:
American Concrete Pressure Pipe Assn.
Cast Iron Pipe Research Assn.
Steel Plate Fabricators Assn.

Brass Goods:
Anaconda American Brass Co.
Hays Mfg. Co.
Mueller Co.

Brine-Making Equipment:
International Salt Co., Inc.

Calcium Hypochlorite:
Jones Chemicals, Inc.
Olin Mathieson Chemical Corp.

Carbon Dioxide Generators:
Ozark-Mahoning Co.
Walker Process Equipment, Inc.

Cathodic Protection:
Electro Rust-Proofing Corp.
Harco Corp.

Cement Mortar Lining:
Centriline Corp.
Halliburton Co.
Southern Pipe Div. of U.S. Industries

Chemical Feed Apparatus:
B-I-F Industries, Inc.—Omega
B-I-F Industries, Inc.—Proportioneers
Fischer & Porter Co.
F. B. Leopold Co.
Permutit Co.
Precision Chemical Pump Corp.
Simplex Valve & Meter Co.
Wallace & Tiernan Inc.

Chemists and Engineers:
(See Professional Services)

Chlorination Equipment:
B-I-F Industries, Inc.—Builders
B-I-F Industries, Inc.—Proportioneers
Fischer & Porter Co.
Precision Chemical Pump Corp.
Wallace & Tiernan Inc.

Chlorine Comparators:
Fischer & Porter Co.
Klett Mfg. Co.
Wallace & Tiernan Inc.

Chlorine, Liquid:
Jones Chemicals, Inc.
Olin Mathieson Chemical Corp.
Wallace & Tiernan Inc.

Clamps and Sleeves, Pipe:
James B. Clow & Sons
Dresser Mfg. Div.
Mueller Co.
A. P. Smith Mfg. Co.
Trinity Valley Iron & Steel Co.

Clamps, Bell Joint:
James B. Clow & Sons
Dresser Mfg. Div.

Clamps, Pipe Repair:
James B. Clow & Sons
Dresser Mfg. Div.
Trinity Valley Iron & Steel Co.

Clarifiers:
American Well Works
Dorr-Oliver Inc.
Eimco Corp., The
General Filter Co.
Infilco Inc.
Permutit Co.
Walker Process Equipment, Inc.

COLD WATER COAGULATING PROBLEMS • Chemical reactions are slower • Water gravity & viscosity increase • Alkalinity may drop

Activated silica sol, made by reacting PQ "N" sodium silicate, is an acknowledged aid wherever water coagulating problems occur. Troublesome waters particularly, may require special techniques to form a sizable floc. We shall be glad to discuss these with you.

Our activated silica sol experience dates to the late 1930's. PQ silica sol developments are known as the N-Sol Processes. Please call on us with your coagulating problems.

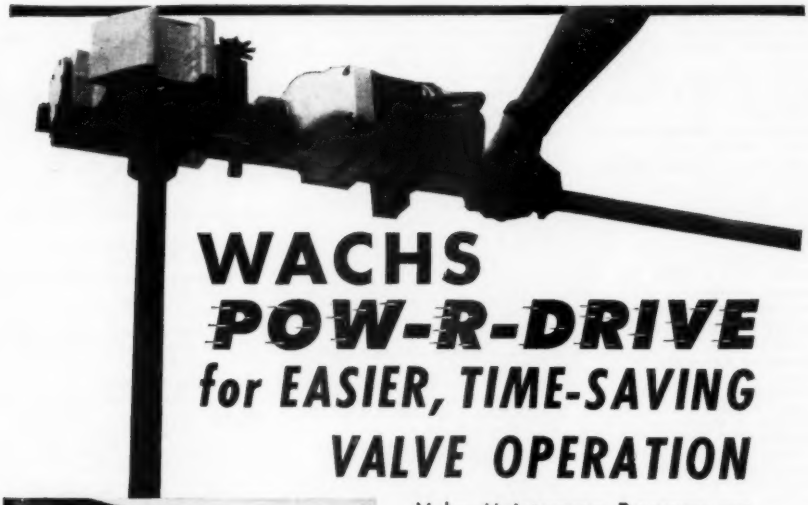
PQ SOLUBLE SILICATES

PHILADELPHIA QUARTZ CO.
1142 Public Ledger Bldg., Philadelphia 6, Pa.

Associates: Philadelphia Quartz Co. of Calif. Berkeley & Los Angeles, Calif.; Tacoma, Wash.; National Silicates Limited, Toronto & Valleyfield, Canada

1831—130TH ANNIVERSARY—1961

PQ Plants: Anderson, Ind.; Baltimore, Md.; Buffalo, N.Y.; Chester, Pa.; Jeffersonville, Ind.; Kansas City, Kan.; Rahway, N.J.; St. Louis, Mo.; Utica, Ill.



WACHS POW-R-DRIVE for EASIER, TIME-SAVING VALVE OPERATION

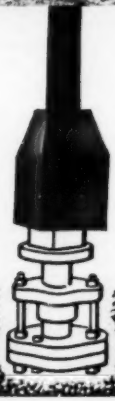
Valve Maintenance Programs are now possible at reasonable cost with Pow-R-Drive. Also gives faster valve operation in emergencies.

Heavy Duty, Portable Pow-R-Drive is ruggedly built to stand up in the toughest field jobs. It can be used in the most inaccessible places and delivers ample power to open and close large valves with great saving of time and labor.

Reversible air or electric drive motors make short work of "Freeing-up" valves.

Other uses

- Operates pipe tappers
- Operates hand winches
- Powers a geared die head for threading or cutting
- Drills horizontal or vertical holes with an auger
- Drills anode holes and post holes
- Operates sluice gates



WACHS CHICAGO

THE E. H. WACHS COMPANY

MANUFACTURERS OF PRECISION MACHINERY SINCE 1883

1525 NORTH DAYTON STREET • CHICAGO 22, ILLINOIS

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Hagan Chemicals & Controls, Inc.
Nalco Chemical Co.
Philadelphia Quartz Co.

Condensers:
Allis-Chalmers Mfg. Co.
United States Pipe & Foundry Co.
Worthington Corp.

Contractors, Water Supply:
Layne & Bowler, Inc.

Controllers, Liquid Level, Rate of Flow:
Bailey Meter Co.
B-I-F Industries, Inc.—Builders
Fischer & Porter Co.
Foxboro Co.
General Filter Co.
Simplex Valve & Meter Co.

Copper Sheets:
Anaconda American Brass Co.

Copper Sulfate:
Tennessee Corp.

Corrosion Control:
Calgon Co.
Harco Corp.
Nalco Chemical Co.
Philadelphia Quartz Co.
Southern Pipe Div. of U.S. Industries

Couplings, Flexible:
Dresser Mfg. Div.
Southern Pipe Div. of U.S. Industries

Covers, Vault:
Ford Meter Box Co.
Wachs, E. H., Co.

Diaphragms, Pump:
Dorr-Oliver Inc.

Engineers and Chemists:
(See Professional Services)

Evaporating Equipment:
Ozark-Mahoning Co.

Excavating Equipment:
Charles Machine Works, Inc.
Eimco Corp., The

Feedwater Treatment:
B-I-F Industries, Inc.—Proportioneers
Calgon Co.
Hungerford & Terry, Inc.
Nalco Chemical Co.
Permutit Co.

Ferric Sulfate:
Tennessee Corp.

Filter Materials:
Anthracite Equipment Corp.
Carborundum Co.
General Filter Co.
Johns-Manville Corp.
Multi-Metal Wire Cloth Co., Inc.
Northern Gravel Co.
Permutit Co.
Stuart Corp.

Filters, incl. Feedwater:
B-I-F Industries, Inc.—Proportioneers
Dorr-Oliver Inc.
Eimco Corp., The
Permutit Co.
Roberts Filter Mfg. Co.
Ross Valve Mfg. Co.

Filtration Plant Equipment:
B-I-F Industries, Inc.—Builders
B-I-F Industries, Inc.—Omega
Chain Belt Co.
Eimco Corp., The
Filtration Equipment Corp.
General Filter Co.
Golden-Anderson Valve Specialty Co.
Hungerford & Terry, Inc.
Infilco Inc.
F. B. Leopold Co.
Multi-Metal Wire Cloth Co., Inc.
Permutit Co.
Roberts Filter Mfg. Co.
Simplex Valve & Meter Co.
Stuart Corp.
Wallace & Tiernan Inc.

Fittings, Copper Pipe:
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Hays Mfg. Co.
Mueller Co.

Fittings, Tees, Ells, etc.:
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James B. Clow & Sons
Dresser Mfg. Div.
M & H Valve & Fittings Co.
Morgan Steel Products, Inc.
Southern Pipe Div. of U.S. Industries
Trinity Valley Iron & Steel Co.
United States Pipe & Foundry Co.
R. D. Wood Co.

Flocculating Equipment:
Chain Belt Co.
Dorr-Oliver Inc.
Eimco Corp., The
General Filter Co.
Infilco Inc.
F. B. Leopold Co.
Permutit Co.
Stuart Corp.

Fluoride Chemicals:
American Agricultural Chemical Co.
General Chemical Div., Allied Chemical Corp.
Olin Mathieson Chemical Corp.
Ozark-Mahoning Co.
Tennessee Corp.

Fluoride Feeders:
B-I-F Industries, Inc.—Omega
B-I-F Industries, Inc.—Proportioneers
Fischer & Porter Co.
Wallace & Tiernan Co., Inc.

Furnaces:
Jos. G. Pollard Co., Inc.

Gages, Liquid Level:
Bailey Meter Co.
B-I-F Industries, Inc.—Builders
Fischer & Porter Co.
Simplex Valve & Meter Co.
Wallace & Tiernan Inc.

Gages, Loss of Head, Pressure of Vacuum, Rate of Flow, Sand Expansion:
Bailey Meter Co.
B-I-F Industries, Inc.—Builders
Fischer & Porter Co.
Foxboro Co.
Jos. G. Pollard Co., Inc.
Simplex Valve & Meter Co.
Wallace & Tiernan Inc.

Gasholders:
Bethlehem Steel Co.
Chicago Bridge & Iron Co.
Graver Tank & Mfg. Co.
Pittsburgh-Des Moines Steel Co.

Gaskets, Rubber Packing:
James B. Clow & Sons
Johns-Manville Corp.

Gates, Shear and Sluice:
Armco Drainage & Metal Products, Inc.
James B. Clow & Sons
Mueller Co.
R. D. Wood Co.

Gears, Speed Reducing:
DeLaval Steam Turbine Co.
Worthington Corp.

Glass Standards—Colorimetric Analysis Equipment:
Fischer & Porter Co.
Klett Mfg. Co.
Wallace & Tiernan Inc.

Goosenecks (with or without Corporation Stops):
James B. Clow & Sons
Mueller Co.
Southern Pipe Div. of U.S. Industries

Hydrants:
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Darling Valve & Mfg. Co.
M. Greenberg's Sons
Kennedy Valve Mfg. Co.
M & H Valve & Fittings Co.
Mueller Co.
A. P. Smith Mfg. Co.
R. D. Wood Co.

Hydrogen Ion Equipment:
Photovolt Corp.
Wallace & Tiernan Inc.

Hypochlorite; see Calcium Hypochlorite; Sodium Hypochlorite

Intake Screens:
Multi-Metal Wire Cloth Co., Inc.

Ion Exchange Materials:
General Filter Co.
Hungerford & Terry, Inc.
Nalco Chemical Co.
Permutit Co.
Roberts Filter Mfg. Co.
Rohm & Haas Co.

Iron, Pig:
Woodward Iron Co.

Iron Removal Plants:
American Well Works
General Filter Co.
Hungerford & Terry, Inc.
Permutit Co.
Roberts Filter Mfg. Co.
Walker Process Equipment, Inc.

Jointing Materials:
Johns-Manville Corp.
Keasbey & Mattison Co.
Leadite Co., Inc.

Joints, Mechanical, Pipe:
American Cast Iron Pipe Co.
James B. Clow & Sons
Dresser Mfg. Div.
Southern Pipe Div. of U.S. Industries
Trinity Valley Iron & Steel Co.
United States Pipe & Foundry Co.
R. D. Wood Co.

Leak Detectors:
Aqua Survey & Instrument Co.
Jos. G. Pollard Co., Inc.

Lime Slakers and Feeders:
B-I-F Industries, Inc.—Omega
Dorr-Oliver Inc.
General Filter Co.
Infilco Inc.
Permutit Co.
Wallace & Tiernan Inc.

Locators, Pipe & Valve Box:
Aqua Survey & Instrument Co.
W. S. Darley & Co.
Jos. G. Pollard Co., Inc.

Magnetic Dipping Needles:
Aqua Survey & Instrument Co.
W. S. Darley & Co.

Meter Boxes:
Ford Meter Box Co.
Rockwell Mfg. Co.

Meter Couplings and Yokes:
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Dresser Mfg. Div.
Ford Meter Box Co.
Gamon Meter Div., Worthington Corp.

longer life

PATENTED SILICONE STUFFING BOX PACKING

constantly improved

parts always available

WATCH DOG

Split case or frost proof

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WORTHINGTON

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Mueller Co.
Neptune Meter Co.
Rockwell Mfg. Co.

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Badger Meter Mfg. Co.

Meter Testers:
Badger Meter Mfg. Co.
Ford Meter Box Co.
Hersey-Sparling Meter Co.
Neptune Meter Co.
Rockwell Mfg. Co.

Meters, Domestic:
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Buffalo Meter Co.
Calmet Meter Div., Worthington Corp.
Gamon Meter Div., Worthington Corp.
Hersey-Sparling Meter Co.
Neptune Meter Co.
Rockwell Mfg. Co.

Meters, Filtration Plant, Pumping Station, Transmission Line:
Bailey Meter Co.
B-I-F Industries, Inc.—Builders
Fischer & Porter Co.
Simplex Valve & Meter Co.

Meters, Industrial, Commercial:
Badger Meter Mfg. Co.
Bailey Meter Co.
B-I-F Industries, Inc.—Builders
Buffalo Meter Co.
Calmet Meter Div., Worthington Corp.
Fischer & Porter Co.
Gamon Meter Div., Worthington Corp.
Hersey-Sparling Meter Co.
Neptune Meter Co.
Rockwell Mfg. Co.
Simplex Valve & Meter Co.

Mixing Equipment:
General Filter Co.
F. B. Leopold Co.

Motors, Electric:
Allis-Chalmers Mfg. Co.
Marathon Electric Mfg. Corp.
Worthington Corp.

Paints:
Inertol Co., Inc.
Koppers Co., Inc.
Plastics & Coal Chemicals Div., Allied Chemical Corp.

Pipe, Asbestos-Cement:
Atlas Asbestos Co. Ltd.
Johns-Manville Corp.
Keasbey & Mattison Co.

Pipe, Brass:
Anaconda American Brass Co.

Pipe, Cast Iron (and Fittings):
Alabama Pipe Co.
American Cast Iron Pipe Co.
James B. Clow & Sons
Trinity Valley Iron & Steel Co.
United States Pipe & Foundry Co.
R. D. Wood Co.

Pipe, Cement Lined:
American Cast Iron Pipe Co.
James B. Clow & Sons
Southern Pipe Div. of U.S. Industries
United States Pipe & Foundry Co.
R. D. Wood Co.

Pipe, Concrete:
American Pipe & Construction Co.
Lock Joint Pipe Co.
Vulcan Materials Co.

Pipe, Copper:
Anaconda American Brass Co.

Pipe, Plastic:
American Hard Rubber Co.
Keasby & Mattison Co.
Morgan Steel Products, Inc.
Orangeburg Mfg. Co., Div. of The Flintkote Co.

Pipe, Steel:
Armco Drainage & Metal Products, Inc.
Bethlehem Steel Co.
Morgan Steel Products, Inc.
Southern Pipe Div. of U.S. Industries

Pipe Cleaning Services:
Ace Pipe Cleaning, Inc.
Centriline Corp.
National Power Rodding Corp.
National Water Main Cleaning Co.
Robinson Pipe Cleaning Co.

Pipe Coatings and Linings:
American Cast Iron Pipe Co.
American Hard Rubber Co.
Centriline Corp.
Inertol Co., Inc.
Koppers Co., Inc.
Pipe Linings, Inc.
Plastics & Coal Chemicals Div., Allied Chemical Corp.
Reilly Tar & Chemical Corp.
Southern Pipe Div. of U.S. Industries

Pipe Cutters:
James B. Clow & Sons
Ellis & Ford Mfg. Co.
Pilot Mfg. Co.
Jos. G. Pollard Co., Inc.
A. P. Smith Mfg. Co.
Wachs, E. H., Co.
Wheeler Mfg. Corp.

Pipe Jointing Materials; see Jointing Materials

Pipe Locators; see Locators, Pipe

Plugs, Removable:
James B. Clow & Sons
Jos. G. Pollard Co., Inc.
A. P. Smith Mfg. Co.

Potassium Permanganate:
Carus Chemical Co.

Pressure Regulators:
Allis-Chalmers Mfg. Co.
Golden-Anderson Valve Specialty Co.
Mueller Co.
Ross Valve Mfg. Co.

Pumps, Boiler Feed:
DeLaval Steam Turbine Co.

Pumps, Centrifugal:
Allis-Chalmers Mfg. Co.
American Well Works
DeLaval Steam Turbine Co.
Peerless Pump Div.
C. H. Wheeler Mfg. Co.

Pumps, Chemical Feed:
B-I-F Industries, Inc.—Proportioneers
Fischer & Porter Co.
Precision Chemical Pump Corp.
Wallace & Tiernan Inc.

Pumps, Deep Well:
American Well Works
Fiese & Firstenberger
Layne & Bowler, Inc.
Peerless Pump Div.

Pumps, Diaphragm:
Dorr-Oliver Inc.
Wallace & Tiernan Inc.

Pumps, Hydrant:
W. S. Darley & Co.
Jos. G. Pollard Co., Inc.

Pumps, Hydraulic Booster:
Peerless Pump Div.
Ross Valve Mfg. Co.

Pumps, Sewage:
Allis-Chalmers Mfg. Co.
DeLaval Steam Turbine Co.
Peerless Pump Div.
C. H. Wheeler Mfg. Co.

Pumps, Sump:
DeLaval Steam Turbine Co.
Peerless Pump Div.
C. H. Wheeler Mfg. Co.

Pumps, Turbine:
Fiese & Firstenberger
Layne & Bowler, Inc.
Peerless Pump Div.

Recorders, Gas Density, CO_2, NH_3, SO_2, etc.:
Fischer & Porter Co.
Permutit Co.
Wallace & Tiernan Inc.

Recording Instruments:
Bailey Meter Co.
B-I-F Industries, Inc.—Builders
Fischer & Porter Co.
Simplex Valve & Meter Co.
Wallace & Tiernan Inc.

Reservoirs, Steel:
Bethlehem Steel Co.
Chicago Bridge & Iron Co.
Graver Tank & Mfg. Co.
Pittsburgh-Des Moines Steel Co.

Sand Expansion Gages; see Gages

Sleeves: see Clamps

Sleeves and Valves, Tapping:
James B. Clow & Sons
M & H Valve & Fittings Co.
Mueller Co.
Rensselaer Valve Co.
A. P. Smith Mfg. Co.

Sludge Blanket Equipment:
Eimco Corp., The
General Filter Co.
Infilco Inc.
Permutit Co.

Sodium Aluminate:
Nalco Chemical Co.

Sodium Chloride:
International Salt Co., Inc.

Sodium Fluoride:
American Agricultural Chemical Co.
General Chemical Div., Allied Chemical Corp.

Sodium Hexametaphosphate:
Calgon Co.

Sodium Hypochlorite:
Jones Chemicals, Inc.
Wallace & Tiernan Inc.

Sodium Silicate:
General Chemical Div., Allied Chemical Corp.
Philadelphia Quartz Co.

Sodium Silicofluoride:
American Agricultural Chemical Co.
General Chemical Div., Allied Chemical Corp.
Tennessee Corp.

Softeners:
Dorr-Oliver Inc.
General Filter Co.
Hungerford & Terry, Inc.
Permutit Co.
Roberts Filter Mfg. Co.
Walker Process Equipment, Inc.

Softening Chemicals and Compounds:
Calgon Co.
General Filter Co.
International Salt Co., Inc.
Nalco Chemical Co.
Permutit Co.
Tennessee Corp.

Standpipes, Steel:
Bethlehem Steel Co.
Chicago Bridge & Iron Co.
Graver Tank & Mfg. Co.
Pittsburgh-Des Moines Steel Co.

SELF-EXPANDING CORK

THE JOINT MATERIAL THAT "GROWS" AFTER COMPRESSION
SELF-EXPANDING CORK IN JOINT
Normal Opening
JOINT COMPRESSED
CONTRACTED JOINT
Self-Expanding Cork Filler Expands Up To 50% Beyond Original Thickness

...keeps joints filled under maximum contraction

Specify and use Servicised SELF-EXPANDING CORK premolded joint filler in water and sewage treatment plants, canal linings and structures, outlet works, spillways and stilling basins in dams—and any other project where it is essential to keep the joints filled when contraction may open them up to more than original size.

Self-Expanding Cork Joint is formed from clean, granulated cork particles securely bonded together by an insoluble synthetic resin binder. It is specially treated to expand as much as 50% beyond original thickness. Fully compressible, non-extruding and resilient, Servicised Self-Expanding Cork is available in ½", ¾" and 1" thicknesses and lengths up to 10 ft.

Write for your copy of the new Servicised Catalog. It contains complete information on Self-Expanding Cork, as well as many other types of premolded joint fillers, and the Servicised line of hot and cold applied joint sealers we manufacture.

See our Catalog in Sweets.

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6051 West 65th Street **Chicago 38, Illinois**

Steel Plate Construction:
Bethlehem Steel Co.
Chicago Bridge & Iron Co.
Graver Tank & Mfg. Co.
Morgan Steel Products, Inc.
Pittsburgh-Des Moines Steel Co.

Stops, Curb and Corporation:
Ford Meter Box Co.
Hays Mfg. Co.
Mueller Co.

Storage Tanks: see Tanks

Strainers, Suction:
James B. Clow & Sons
R. D. Wood Co.

Surface Wash Equipment:
Golden-Anderson Valve Specialty Co.
Permutit Co.

Swimming Pool Sterilization:
B-I-F Industries, Inc.—Builders
B-I-F Industries, Inc.—Omega
B-I-F Industries, Inc.—Proportioneers
Fischer & Porter Co.
Wallace & Tiernan Inc.

Tank Painting and Repair:
Koppers Co., Inc.
National Tank Maintenance Corp.

Tanks, Prestressed Concrete:
Preload Co., Inc.

Tanks, Steel:
Bethlehem Steel Co.
Chicago Bridge & Iron Co.
Graver Tank & Mfg. Co.
Morgan Steel Products, Inc.
Pittsburgh-Des Moines Steel Co.

Tapping-Drilling Machines:
Hays Mfg. Co.
Mueller Co.
A. P. Smith Mfg. Co.

Tapping Machines, Corp.:
Hays Mfg. Co.
Mueller Co.

Taste and Odor Removal:
B-I-F Industries, Inc.—Builders
B-I-F Industries, Inc.—Proportioneers
General Filter Co.
Industrial Chemical Sales Div.
Permutit Co.
Wallace & Tiernan Inc.

Turbidimetric Apparatus (For Turbidity and Sulfate Determinations):
Wallace & Tiernan Inc.

Turbines, Steam:
Allis-Chalmers Mfg. Co.
DeLaval Steam Turbine Co.

Valve Boxes:
James B. Clow & Sons
Ford Meter Box Co.
M & H Valve & Fittings Co.
Mueller Co.
Rockwell Mfg. Co.
A. P. Smith Mfg. Co.
Trinity Valley Iron & Steel Co.
R. D. Wood Co.

Valve-Inserting Machines:
Mueller Co.
A. P. Smith Mfg. Co.

Valve-Operating Units:
B-I-F Industries, Inc.
Filtration Equipment Corp.
Wachs, E. H., Co.
Wheeler, C. H., Mfg. Co.

Valves, Altitude:
Allis-Chalmers Mfg. Co., Hydraulic Div.
Golden-Anderson Valve Specialty Co.
Ross Valve Mfg. Co., Inc.

Valves, Butterfly, Check, Flap, Foot, Hose, Mud and Plug:
Allis-Chalmers Mfg. Co., Hydraulic Div.
B-I-F Industries, Inc.—Builders
James B. Clow & Sons
DeZurik Corp.
Kennedy Valve Mfg. Co.
M & H Valve & Fittings Co.
Mueller Co.
Pelton Div., Baldwin-Lima-Hamilton
Henry Pratt Co.
Rockwell Mfg. Co.
R. D. Wood Co.

Valves, Detector Check:
Hersey-Sparling Meter Co.

Valves, Electrically Operated:
Allis-Chalmers Mfg. Co., Hydraulic Div.
B-I-F Industries, Inc.—Builders
James B. Clow & Sons
Darling Valve & Mfg. Co.
Golden-Anderson Valve Specialty Co.
Kennedy Valve Mfg. Co.
M & H Valve & Fittings Co.
Mueller Co.
Henry Pratt Co.
Rockwell Mfg. Co.
A. P. Smith Mfg. Co.

Valves, Float:
James B. Clow & Sons
Golden-Anderson Valve Specialty Co.
Henry Pratt Co.
Rockwell Mfg. Co.
Ross Valve Mfg. Co., Inc.

Valves, Gate:
James B. Clow & Sons
Darling Valve & Mfg. Co.
Dresser Mfg. Div.
Kennedy Valve Mfg. Co.
M & H Valve & Fittings Co.
Mueller Co.
A. P. Smith Mfg. Co.
R. D. Wood Co.

Valves, Hydraulically Operated:
Allis-Chalmers Mfg. Co., Hydraulic Div.
B-I-F Industries, Inc.—Builders
James B. Clow & Sons
Darling Valve & Mfg. Co.
DeZurik Corp.
Golden-Anderson Valve Specialty Co.
Kennedy Valve Mfg. Co.
F. B. Leopold Co.
M & H Valve & Fittings Co.
Mueller Co.
Pelton Div., Baldwin-Lima-Hamilton
Henry Pratt Co.
Rockwell Mfg. Co.
A. P. Smith Mfg. Co.
R. D. Wood Co.

Valves, Large Diameter:
Allis-Chalmers Mfg. Co., Hydraulic Div.
James B. Clow & Sons
Darling Valve & Mfg. Co.
Golden-Anderson Valve Specialty Co
Kennedy Valve Mfg. Co.
M & H Valve & Fittings Co.
Mueller Co.
Pelton Div., Baldwin-Lima-Hamilton
Henry Pratt Co.
Rockwell Mfg. Co.
A. P. Smith Mfg. Co.
R. D. Wood Co.

Valves, Regulating:
Allis-Chalmers Mfg. Co., Hydraulic Div.
DeZurik Corp.
Golden-Anderson Valve Specialty Co.
Mueller Co.
Henry Pratt Co.
Rockwell Mfg. Co.
Ross Valve Mfg. Co.

Valves, Swing Check:
James B. Clow & Sons
Darling Valve & Mfg. Co.
Golden-Anderson Valve Specialty Co
M & H Valve & Fittings Co.
Mueller Co.
Rockwell Mfg. Co.
A. P. Smith Mfg. Co.
R. D. Wood Co.

Venturi Tubes:
B-I-F Industries, Inc.—Builders
Rockwell Mfg. Co.
Simplex Valve & Meter Co.

Waterproofing:
Inertol Co., Inc.
Koppers Co., Inc.
Plastics & Coal Chemicals Div., Allied Chemical Corp.

Water Softening Plants; see Softeners

Water Supply Contractors:
Layne & Bowler, Inc.

Water Testing Apparatus:
LaMotte Chem. Products Co.
Wallace & Tiernan Inc.

Water Treatment Plants:
American Well Works
Chain Belt Co.
Chicago Bridge & Iron Co.
Dorr-Oliver Inc.
Eimco Corp., The
General Filter Co.
Hungerford & Terry, Inc.
Infilco Inc.
Permutit Co.
Pittsburgh-Des Moines Steel Co.
Roberts Filter Mfg. Co.
Walker Process Equipment, Inc.
Wallace & Tiernan Inc.

Well Drilling Contractors:
Layne & Bowler, Inc.

Well Reconditioning and Formation Testing:
Halliburton Co.
Layne & Bowler, Inc.

Wrenches, Ratchet:
Dresser Mfg. Div.

Zeolite: see Ion Exchange Materials

A complete Buyers' Guide to all water works products and services offered by AWWA Associate Members appears in the 1959 AWWA Directory.

NIGHT AND DAY

twenty-four hour service

PUMPS • WATER WELLS • WATER TREATMENT

Layne knows that an unfailing supply of water is vital, whether it be for municipal, industrial or agricultural use. With Associate Companies located throughout the world, Layne can provide the speed, the trained technicians and the quality parts to remedy your water problem. Over 75 years of experience accounts for the ever increasing list of customers who depend on Layne for service. For service night and day, call the Man From Layne.

Write for Layne Service Bulletin 100.

LAYNE OFFERS COMPLETE WATER SERVICE: Initial Surveys • explorations • recommendations, site selection foundation and soil-sampling • well drilling • well casing and screen • pump design, manufacture and installation • construction of water systems • maintenance and service • chemical treatment of water wells • water treatment—all backed by Layne Research • Layne services do not replace, but coordinate with the services of consulting, plant and city engineers.

Layne SINCE 1882 COMPLETE WATER SERVICE

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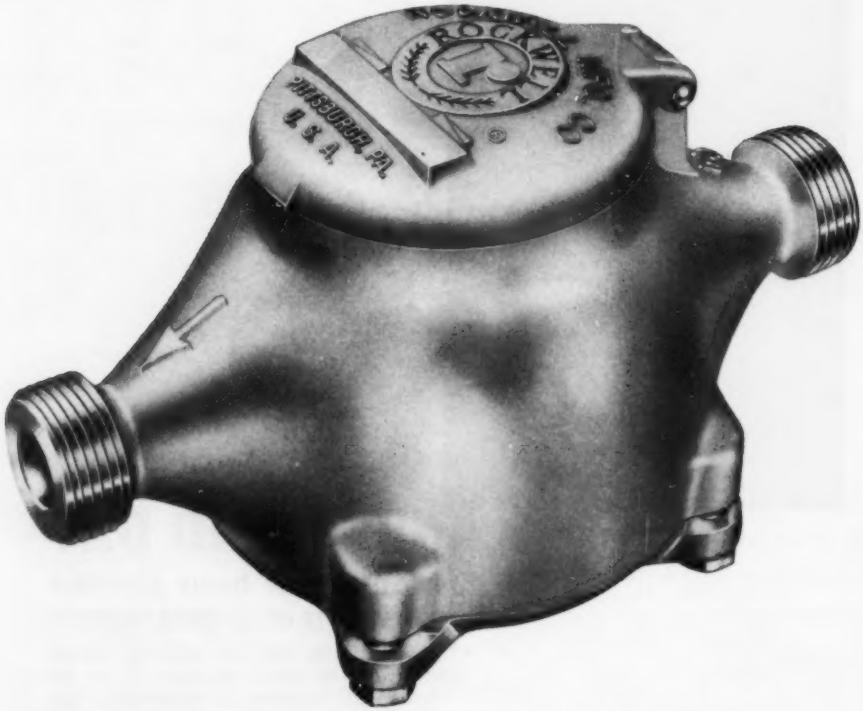
Sales Representatives in Major Cities

Ask the men who use them

(there are many in your area)



Rockwell Sealed Register* "Magnetic" meters

Ask nearby water utilities for *their* experience using Rockwell Sealed Register *magnetic* meters. We'll bank on your getting an enthusiastic testimonial from actual buyers and users.

Hundreds of thousands of these "magnetic" meters are now in service.

*Trade Mark

Please accept this challenge to "ask the men who use them." Write us and we will be glad to send you a list. Rockwell Manufacturing Company, Dept. 163B, Pittsburgh 8, Pa. In Canada: Rockwell Manufacturing Company of Canada, Ltd., Box 420, Guelph, Ontario.

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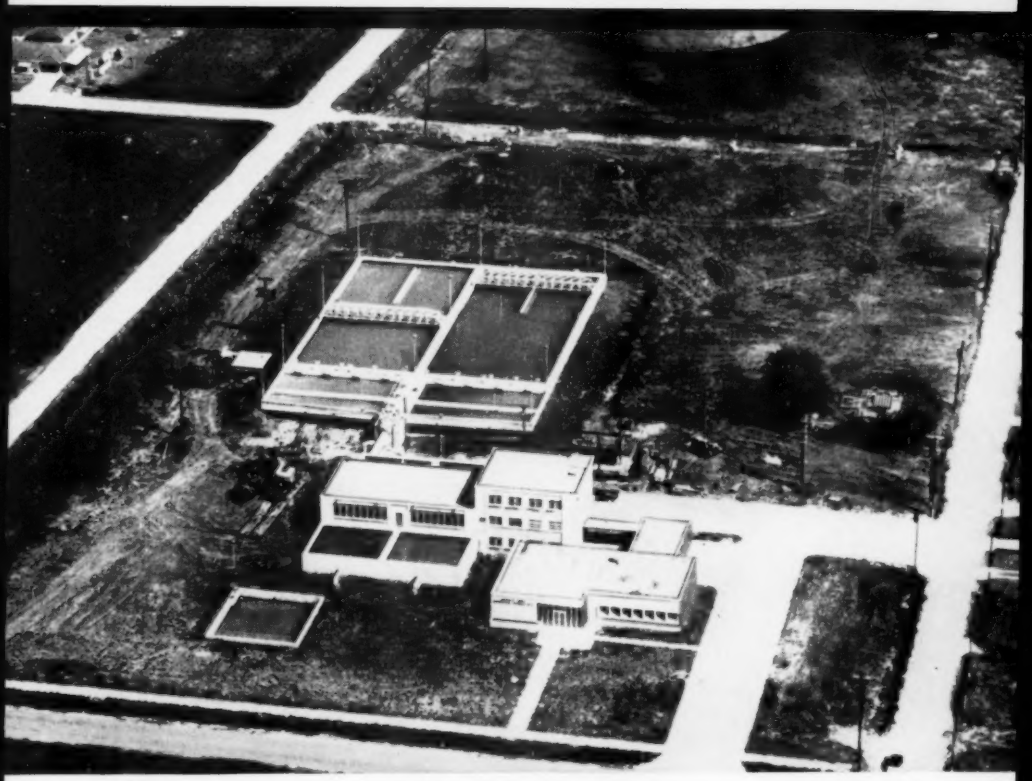

Marrero, Louisiana...

Showplace of modern water treatment practices

To provide for current needs and to prepare for future industrial growth, the City of Marrero, Louisiana, in Jefferson Parish Water Works District No. 2, has put into operation a new 10 mgd water purification plant. With its modern Dorr-Oliver equipment, the installation is a showplace of up-to-the-minute water treatment practices. Equipment includes four Size D Dorrco Flash Mixers, one Dorrco Flocculator for a tank 130′ long and 32′6″ wide with dry well in center, and two Dorrco Monorakes, each for 65′ wide x 130′ long x 15′2″ s.w.d. basins.

For information on the full line of modern Dorr-Oliver water treatment equipment, write to Dorr-Oliver Incorporated, Stamford, Connecticut.

Consulting Engineer: T. Edward Ernst, New Orleans, La.
Contractor: Keller Construction Co., New Orleans, La.
Dorrco, Flocculator, Monorake, T.M Reg. U. S. Pat. Off.

DORR-OLIVER

WORLD-WIDE RESEARCH • ENGINEERING • EQUIPMENT

Three County Commissioners from Ohio report:

"For economy and performance, Transite Water Pipe is still our main choice."

"Belmont was one of the many counties that experienced a building and population boom. Fortunately, our officials had the foresight to recognize its ultimate effect on our water system and service. As early as 1953, plans were made to meet future demands. Surveys were made . . . operating men and engineers were consulted . . . pipe materials investigated.

"In 1956, we extended our water system 13 miles. The installation and operating economies are now a matter of record. The successful performance of the extension is attributed to careful planning, helpful advice and, in part, to the selection of Transite Pipe.

Belmont County, Ohio, Commrs. William H. Dorsey, Austin C. Furbee and Louis T. Salvador.

"When we began designing another expansion of the system for 1960, our previous experience made Transite the main choice. The Belmont Water System now has 53 miles of Transite installed in rocky terrain and corrosive soils. The excellent performance of the first 13-mile section leads us to believe that Transite will provide economical maintenance and operation for many years."

For the full Transite® story, write Johns-Manville, Box 14, JA-2, New York 16, N. Y. In Canada: Port Credit, Ontario. Cable address: Johnmanvil.

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THE WHITE PIPE THAT PROTECTS PRICELESS WATER

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